CA2 AL EN752
86 R86-1

REPORT #
RRTAC 86-1

Critical Analysis Of Settling Pond Design And Alternative Technologies

LEGISLATURE LIBRARY
JUN 23 1986
EDMONTON

THE COAL
ASSOCIATION
OF CANADA

Alberta
LAND CONSERVATION AND
RECLAMATION COUNCIL
Reclamation Research
Technical Advisory Committee

Heritage Fund

Report #
RRTAC 86-1

CRITICAL ANALYSIS OF
SETTLING POND DESIGN
AND
ALTERNATIVE TECHNOLOGIES

Prepared for

The Coal Association of Canada

and

The Reclamation Research Technical Advisory Committee

of

The Land Conservation and Reclamation Council

March 1986

Monenco Consultants Ltd.
400 Monenco Place
801-6th Avenue S.W
CALGARY, Alberta
T2P 3W5

ALBERTA'S RECLAMATION RESEARCH PROGRAM

The regulation of surface distrubances in Alberta is the responsibility of the Land Conservation and Reclamation Council. The Council executive consists of a chairman from the Department of the Environment and two deputy chairmen from the Department of Energy and Natural Resources. Among other functions, the Council oversees programs for reclamation of abandoned disturbances and reclamation research. The reclamation research program was established to provide answers to the many practical questions which arise in reclamation. Funds for implementing both the operational and research programs are drawn from Alberta's Heritage Savings Trust Fund.

To assist in technical matters related to the development and administration of the research program the Council appointed the Reclamation Research Technical Advisory Committee (RRTAC). The Committee first met in March, 1978 and consists of eight members representing the Alberta Departments of Agriculture, Energy and Natural Resources, Environment and the Alberta Research Council. The Committee meets regularly to update research priorities, review solicited and unsolicited research proposals, arrange workshops and otherwise act as a referral and coordinating body for Reclamation Research.

Additional information on the Reclamation Research Program may be obtained by contacting:

Dr. P.F. Ziemkiewicz, Chairman
Reclamation Research Technical Advisory Committee
Alberta Energy and Natural Resources
2nd Floor, 10909 Jasper Avenue
EDMONTON, Alberta
T5J 3M8 (427-8042)

This Report may be cited as:

Monenco Consultants Ltd. 1986.
Critical Analysis of Settling Pond Design and Alternative Technologies. The Coal Association of Canada and Alberta Land Conservation and Reclamation Council
Report #RRTAC 86-1. 372 p.

Additional Copies may be obtained from:

Publication Services
Queen's Printer
11510 Kingsway Avenue
EDMONTON, Alberta
T5G 2Y5

This report contains the results of a study jointly financed by The Coal Association of Canada and The Alberta Land Conservation and Reclamation Council. The project was managed by The Hydrology Subcommittee of the Industry/Government Mountain and Foothill Coal Reclamation Research Program.

The opinions, findings, conclusions or recommendations expressed in this report are those of the author(s) and do not necessarily reflect the views of The Coal Association of Canada and the Alberta Government. Mention of trade names or commercial products does not constitute endorsement or recommendation for use by The Coal Association of Canada or the Alberta Government.

EXECUTIVE SUMMARY

There has been increasing concern in recent years regarding the environmental impact of sediment laden runoffs discharged from land disturbing activities in Alberta. The most commonly used structures for the control of sediment discharges are settling ponds, which are constructed to retain contaminated wastewaters long enough to affect the removal of sediment by gravity settling. This study was undertaken to critically evaluate the various settling pond design methodologies currently available and to provide a catalogue of technologies that might provide an alternative to sediment removal by ponding.

The evaluation of settling pond design methodologies and potential alternative technologies was completed by conducting a comprehensive review of the available literature. This review was completed using both computer and manual searches and was successful in identifying a wide range of information on settling pond design and performance. Descriptions of alternative technologies applied to disturbed area sediment removal were not as plentiful, requiring that potentially suitable alternatives be identified by considering their demonstrated performance in other similar applications.

The following sections outline the principal findings of the literature review and briefly describe the major conclusions drawn with respect to settling pond design and alternative technologies for the Alberta condition. The final section provides recommendations concerning areas requiring further study and settling pond design considerations for the Alberta foothills and front ranges.

1. SETTLING POND DESIGN

The review of settling pond design methods highlighted separate procedures for estimating the required volume for each of the following basic components of total reservoir storage capacity:

1. Sediment storage volume;
2. Permanent pool volume;
3. Detention volume; and
4. Flood storage volume.

Of the above components, the detention volume has the most profound effect on the sediment removal efficiency of a particular impoundment. The detention volume is typically sized by satisfying the requirements of one or more of the following criteria:

1. Sediment removal efficiency;
2. Detention time;
3. Total suspended solids (TSS) concentration;
4. Settleable solids (SS) concentration; and
5. Total sediment discharge.

One of a number of hydrology and/or sedimentology models is used to predict whether or not a particular settling pond configuration has a detention volume which is capable of satisfying the design criteria selected. The various models currently available differ greatly in sophistication and in their ability to evaluate the effects of all those parameters which control sediment transport through a reservoir. The most significant of these controlling variables are:

1. Physical characteristics of the sediment;
2. Hydraulic characteristics of the basin;
3. Inflow sedimentgraph;
4. Inflow hydrograph;
5. Basin geometry;
6. Water and sediment chemistry; and
7. Presence of thermal gradients.

The sedimentology models that are described in this document are summarized in Table E1 along with some of their strengths and weaknesses, particularly with respect to conditions encountered in the Alberta foothills and front ranges. All but the simplest of the models noted in Table E1 make use of outputs from some form of watershed hydrology model. These watershed models are used to predict the hydraulic and sediment loading for a particular pond during a

Table E1. Summary of pond sedimentation models.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
1. Empirical Models	Brune (see pages 85 and 139)	No	- Simple to apply	- Does not account for characteristics of incoming sediment. - Poorly suited to predictions of pond performance during single storm events. - Not suitable for application to small settling ponds. - Does not allow for consideration of the effects of steep slopes and low water temperatures.
	Churchill (see pages 88 and 140)	No	- Easier to apply than most physically based models - Generally provides more accurate predictions of trap efficiency than Brune's method	- Does not account for characteristics of incoming sediment. - Does not consider the effects of low water temperature. - Empirical method based on limited volume of data.

continued...

Table E1. Continued.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
1. Empirical Models (Cont'd)	Ward's Equations (see page 90)	No	- Regression equations generated by computer model which considers the temporal variation in detention time during storm events and the characteristics of the incoming sediment. - Provide better estimates of pond performance than Brune's or Churchill's curves.	- May not provide accurate results for ponds with storm event detention times greater than 10 hours. - Assume a constant water temperature of 20°C. - Do not consider the effects of short circulating, turbulence, and thermal stratification.
2. Physically Based Models				
a. Overflow Rate Models	Steady State Flow: USEPA Model (see pages 95 and 141)	No	- Overflow rate concepts are relatively easy to understand making the model's application straightforward.	- Does not account for the temporal variation in detention time during a storm event or the impact of thermal stratification.

continued...

Table E1. Continued.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
2. Physically Based Models a. Overflow Rate Models (Cont'd)	Steady State Flow: USEPA Model (Cont'd)		- Considers the characteristics of incoming sediment. - Accounts for the impact of low water temperatures on settling rates.	- Accounts for all non-ideal settling with one empirical coefficient. - Accuracy of predicted peak effluent concentrations tends to be erratic.
	Variable Overflow Rate Models (including Tapp's Methods, see pages 96 to 98 and 142)		- Provide advantages of USEPA model plus straightforward considerations of the temporal variation in detention time during a storm event.	- Require more computational effort than the USEPA model. - Account for all non-ideal settling with one empirical coefficient. - Accuracy of predicted peak effluent concentrations generally better than USEPA model but still subject to a high margin of variability.

continued...

Table E1. Continued.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
2. Physically Based Models a. Overflow Rate Models (Cont'd)	Turbulent Flow (see pages 99 and 143)	No	- Provides a procedure for including the effects of turbulence in either steady (USEPA) or unsteady (Tapp's methods) flow overflow rate models. - Computational procedure is fairly straightforward.	- In many cases, the consideration of turbulence with this model does not significantly improve the accuracy of predicted peak effluent concentrations.
b. Reactor Models	DEPOSITS (see pages 113 and 144)	Yes	- Considers the characteristics of the inflow and outflow sediment graphs and hydrographs (i.e. accounts for the temporal variation in sediment loading, hydraulic loading, and detention time).	- Requires relatively high level of computational effort. - Results can be very sensitive to variations in input parameters (particularly sediment particle size distributions and specific gravity).

continued...

Table E1. Continued.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
2. Physically Based Models b. Reactor Models (Cont'd)	DEPOSITS (Cont'd)		- Considers the effects of basin geometry, spillway characteristics, particle size distributions, temperature, permanent pool volume, and density currents. - Provides accurate predictions of trapping efficiency.	- Effects of short circuiting and non-ideal settling are accounted for with empirical factors which are not always easy to define. - Effects of thermal stratification are not considered. - Predictions of the timing and magnitude of peak effluent concentrations have been erratic in tests to date.
	CSTRS (see pages 116 and 146)	Yes	- As per DEPOSITS except that CSTRS models both the mixing and delayed flow characteristics of a pond and therefore provides more accurate	- As per DEPOSITS except that CSTRS provides more accurate predictions of outflow sedimentgraph values. - Results are very sensitive to the number of reactors used

continued...

Table E1. Continued.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
2. Physically Based Models b. Reactor Models (Cont'd)	CSTRS (Cont'd)		predictions of the timing and magnitude of peak effluent concentrations.	for a particular pond. - Procedures for selecting the appropriate number of reactors for new ponds are not well established.
	BASIN (see pages 124 and 147)	Yes	- As per CSTRS except that vertical resuspension of sediment is modelled. - Provides better predictions of outflow sedimentgraph values than CSTRS for ponds with shallow flows and high velocities.	- As per CSTRS. - Model is not presently included within the framework of any available watershed model.

continued...

Table E1. Continued.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
2. Physically Based Models c. Other Models	CREAMS (see pages 137 and 142)	Yes	- Accounts for the effects of storm flow characteristics, sediment characteristics, pond geometry, and outlet chracteristics.	- Applicable only to small tile drained impoundments on the order of 0.2 hectares or less. - Does not consider thermal stratification. - Limited number of verification studies conducted to date show a high margin of error in model outputs.
	FARMPND (see page 136)	In preparation	- Model will be capable of evaluating the impacts of inflow rates and sediment concentrations, pond geometry, particle size distributions, and thermal stratification.	- Quality of predicted outputs and their sensitivity to quality of input data are indeterminate at this time.

continued...

Table E1. Concluded.

Model Type	Model Name	Computer Program Available	Strong Points	Weak Points
2. Physically Based Models c. Other Models (Cont'd)	FARMPND (Cont'd)		- Model will be a continuous simulation model rather than a single storm event model.	

given storm event. The model outputs typically take the form of hydrographs and sedimentgraphs applicable to the inlet of a settling pond. The extent to which a particular sedimentology model uses these outputs varies with its sophistication. For example, the relatively straightforward USEPA overflow rate model uses only the peak pond outflow during a storm event and does not consider the temporal variation in pond detention time. Reactor models on the other hand (i.e. DEPOSITS, CSTRS, BASIN), consider both the inlet and outlet pond hydrographs to account for the variation of detention time during a storm event.

It should be noted that the predictions of all the sedimentology models are extremely sensitive to the margin of variability associated with the hydrology model outputs. For example, the peak effluent sediment concentration predicted by DEPOSITS for one particular pond and watershed varied by 44% for a 10% change in the runoff curve number (Cn), an input parameter for one of the more commonly used hydrology models. This kind of sensitivity illustrates the need for a high level of effort during input data definition if accurate predictions of the timing and magnitude of peak effluent concentrations are required. The use of inputs for hydrology and/or sedimentology models that are poorly correlated to site specific conditions can result in predictions of pond performance that bear little or no resemblance to reality, regardless of the level of sophistication of the models used.

The determination of which sedimentology model is most appropriate for a given design exercise would be site, time, and project specific. It is not necessarily true that the most sophisticated of the models described in Table E1 are always the most suitable. Such factors as project scope, land and information availability, economic constraints, ratio of controlled to uncontrolled flow, and regulatory objectives would have to be reviewed for each proposed settling pond to define the appropriate level of design model sophistication and complexity.

2. ALTERNATIVE TECHNOLOGIES

In evaluating alternatives to sediment removal by ponding, both augmentative and alternative technologies were examined. Augmentative technologies are not intended to replace settling ponds, but act rather to either reduce the sediment load on a pond or to improve the efficiency with which a pond copes with a given sediment load. The erosion reducing augmentative technologies are employed at various locations and stages of the land disturbing activity and often provide the most cost effective solution in situations in which an existing settling pond is incapable of handling the sediment load it receives. However, the efficiency of erosion reducing augmentation technologies is often difficult to maximize unless they have been incorporated into the proposed operational scheme during the planning stages of a project.

Augmentative technologies that improve the efficiency of settling ponds are normally applied at the settling pond site. Two augmentative technologies of this type that are commonly applied with considerable success are polymer flocculants and in-pond baffles. Polymers improve pond efficiency by promoting the agglomeration of small particles into larger, more settleable particles, while baffles can be used to provide similar results by increasing the length of the flow path, and hence the detention time in a pond.

The literature review also identified a number of 'stand alone' alternative technologies that could be used to supplant settling ponds altogether. The technologies identified as exhibiting the highest potential for disturbed area sediment removal applications are listed in Table E2 along with a qualitative rating of each alter-

Table E2. Summary of alternative technologies.

Technology	Potential for Producing Acceptable Effluent from Influents with Fine Particle Size Distributions[a,b]	Ability to Tolerate High Suspended Solids Concentrations while Maintaining Consistent Effluent Quality[a,c,d]	Required Level of Operating and Maintenance Input and Expertise[a]	Space Requirements[a]	Estimated Total Annual Cost for 10 Year Operation[e,f] ($/yr)
1. Chemical Treatment Plant	H	L	H	H	504 000
2. Centrifugation	M	VH	H	M	652 000
3. Vacuum Filtration	M	VH	H	M	311 000
4. Hydrocyclones	L	VH	L	M	66 000
5. Ultrafiltration	VH	L	M	L	623 000
6. Cross-Flow Microfiltration	VH	H	M	L	367 000

continued...

Table E2. Concluded.

Technology	Potential for Producing Acceptable Effluent from Influents with Fine Particle Size Distributions[a,b]	Ability to Tolerate High Suspended Solids Concentrations while Maintaining Consistent Effluent Quality[a,c,d]	Required Level of Operating and Maintenance Input and Expertise[a]	Space Requirements[a]	Estimated Total Annual Cost for 10 Year Operation[e,f] ($/yr)
7. Electrokinetic Methods	VH	VH	H	M	464 000
8. Settling Pond	L	M	VL	VH	130 000

[a] Rated qualitatively on the following scale
VL = Very Low, L = Low, M = Moderate, H = High, VH = Very High

[b] Acceptable effluent quality is assumed to be one with a suspended solids concentration at or below 50 mg/L.

[c] 'High' TSS levels are assumed to be those above approximately 10 000 mg/L.

[d] Effluent quality maintained is not necessarily within compliance limits.

[e] Operating and maintenance charges plus capital costs annualized using sinking fund depreciation @ i = 6% and n = 10 years; all costs expressed in first quarter 1985 Canadian dollars.

[f] Costs are based on wastewater flow and characteristics noted in Table 51 (page 341).

native with respect to a number of relevant considerations. For comparative purposes, the table also includes an evaluation of a settling pond incorporating a polymer feed system.

Table E2 shows that although a number of these alternatives are capable of producing high quality effluents from concentrated and fine grained influents, the costs associated with their operation would be, in almost all cases, considerably higher than those for a settling pond operating in conjunction with a polymer feed system. It can be concluded that most of these alternatives would only be appropriate for situations in which the incoming feedwater has a very high proportion of fine grained particles or areas in which space is not available for settling pond construction. If these kinds of site specific factors dictate against the use of settling ponds, the selection of an alternate sediment control strategy should be completed recognizing that the most cost-effective solution may make use of two or even three alternative technologies, each operating over the range of influent characteristics for which its efficiency is maximized.

3. RECOMMENDATIONS

a. A generic planning methodology should be formulated which would guide the sediment control designer in selecting an appropriate level of settling pond design model sophistication for a particular site and give criteria that could be used to determine when an alternate sediment control technology should be considered. The planning methodology should provide techniques for evaluating the relative influences of project scope, land and information availability, economic constraints, flow conditions, and regulatory objectives on design model and/or alternative technology selection. The methodology should also include procedures for obtaining and interpreting the level of input data appropriate for the design model(s) selected.

b. If a land disturbing activity is conducted over a long period of time and involves the construction of a number of sediment control facilities, a data gathering program should be established after the first installation is put into operation. Such a program would provide the well defined input data that would be necessary to cost effectively design subsequent sediment removal facilities. Some of the more relevant parameters that should be included in such a data gathering program are:
    i. Watershed storm event summaries including rainfall depth, intensity, and duration;
    ii. Influent flow;
    iii. Influent TSS;
    iv. Influent particle size distribution;
    v. Specific gravity of sediment particles;
    vi. Effluent TSS;
    vii. Effluent particle size distribution;
    viii. Sludge depth (settling ponds only);
    ix. Sludge density;
    x. Polymer dosages (if any); and
    xi. Relationship between polymer dosage and influent particle size distribution (if applicable).

c. Settling ponds constructed in the Alberta foothills and front ranges should be designed for the most critical of the following three conditions.
    i. Summer conditions with the 1 in 10 year, 24 hour uncontrolled storm flow using the sediment concentration and particle size distribution associated with that storm (this would only apply if a significant volume of pond inflow is uncontrolled);
    ii. Summer conditions with both the maximum and the most protracted controlled (pumped) flows using the sediment concentrations and particle size distributions

associated with those flows (this would only apply if a significant volume of pond inflow is controlled); and

iii. Winter or spring conditions with the maximum controlled (pumped) winter or spring flow in conjunction with a surface ice cover, reduced water temperatures, and the sediment concentrations and particle size distributions associated with the controlled flow in question.

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

1. INTRODUCTION 1
1.1 Nature and Scope of the Study 1
1.2 Terms of Reference 1
1.3 Literature Search 2

2. SETTLING POND DESIGN 4
2.1 Introduction 4
2.2 Regulatory Objectives 4
2.3 Developing Design Storms 6
2.3.1 Introduction 6
2.3.2 Design Storm 7
2.3.3 Estimation of Rainfall Losses 8
2.3.4 Unit Hydrograph Estimation 9
2.3.5 Computation of the Design Flood Hydrograph 15
2.3.6 Use of Local Data 15
2.4 Design Requirements 15
2.4.1 Sediment Storage Volume (V_s) 15
2.4.2 Permanent Pool Volume (V_p) 15
2.4.3 Detention Volume for Settling (V_d) 18
2.4.4. Flood Storage Volume 18
2.5 Design Approaches 18
2.5.1 Sediment Storage Volume 18

TABLE OF CONTENTS (CONTINUED)

Page

2.5.1.1 Rule of thumb approach 19
2.5.1.2 Gross erosion with sediment delivery ratio 19
2.5.1.2.1 Gross erosion 19
2.5.1.2.2 Delivery ratio 28
2.5.1.3 Sediment yield model 34
2.5.1.4 Reservoir survey method 35
2.5.1.5 Density of deposited material 36
2.5.2 Permanent Pool Volume 37
2.5.3 Detention Volume 38
2.5.3.1 Sediment removal efficiency 38
2.5.3.2 Detention time 40
2.5.3.2.1 Uncontrolled (storm) flow 41
2.5.3.2.2 Controlled flow 50
2.5.3.3 Total suspended solids (TSS) 57
2.5.3.4 Effluent settleable solids 58
2.5.3.5 Total design storm 62
2.5.4 Flood Storage Volume 62
2.6 Sedimentation Processes in Ponds 63
2.6.1 Fundamentals 63
2.6.1.1 Physical characteristics of sediment 63
2.6.1.1.1 Discrete settling (Type 1) 65
2.6.1.1.2 Flocculant settling (Type 2) 69
2.6.1.1.3 Hindered settling (Type 3) 78
2.6.1.1.4 Compression settling (Type 4) 78
2.6.1.1.5 Summary comments on settling 78
2.6.1.2 Water chemistry 78
2.6.1.3 Thermal stratification and density currents 80
2.6.2 Sedimentation Models 84
2.6.2.1 Empirical Models 85
2.6.2.1.1 Brune's curve 85

TABLE OF CONTENTS (CONTINUED)

Page

2.6.2.1.2 Churchill's curve 88
2.6.2.1.3 Ward's equation 90
2.6.2.2 Physically based models: Overflow rate approach 93
2.6.2.2.1 Steady state flow: Camp's method 93
2.6.2.2.2 Steady state flow: USEPA model 95
2.6.2.2.3 Unsteady flow: Curtis and McCuen model 96
2.6.2.2.4 Unsteady flow: Tapp's methods 97
2.6.2.2.5 Turbulent flow 99
2.6.2.2.6 Summary comments on the overflow rate approach .. 102
2.6.2.3 Physical based models: reactor models 102
2.6.2.3.1 Single continuous stirred tank reactor 103
2.6.2.3.2 Series of continuous stirred tank reactors 105
2.6.2.3.3 Plug flow reactor 107
2.6.2.3.4 Short-circuiting and dead space 109
2.6.2.3.5 Estimating the hydraulic response of a pond 111
2.6.2.3.6 The DEPOSITS plug flow reactor model 113
2.6.2.3.7 The CSTRS reactor model 116
2.6.2.3.8 The BASIN reactor diffusion model 124
2.6.2.4 Physically based models: hydrodynamic models ... 132
2.6.2.5 Physically based models: other models 136
2.6.2.5.1 The USDA FARMPND model 136
2.6.2.5.2 The CREAMS impoundment model 137
2.6.2.6 Summary tables for pond model inputs 137
2.7 Complete Watershed Models 138
2.8 Cost Considerations 150
2.8.1 Introduction 150
2.8.2 Watershed Characteristics 161
2.8.3 Common Pond Characteristics 161
2.8.4 Model Input and Output Data 162
2.8.5 Cost Estimates 162

TABLE OF CONTENTS (CONTINUED)

Page

2.9 Case Histories 184
2.9.1 Overflow Rate Models 184
2.9.1.1 Disturbed area description 185
2.9.1.2 Settling pond 185
2.9.1.2.1 Description 185
2.9.1.2.2 Design methodology and criteria 187
2.9.1.2.3 Operational history 188
2.9.1.2.4 Cost 190
2.9.1.2.5 Discussion 190
2.9.2 Reactor Models 191
2.9.2.1 Disturbed area description 192
2.9.2.2 Settling ponds 195
2.9.2.2.1 Description 195
2.9.2.2.2 Original design criteria 195
2.9.2.2.3 Cost 197
2.9.2.3 Design model development and application 197
2.9.2.3.1 Model inputs 197
2.9.2.3.2 Model outputs 197
2.9.2.4 Discussion 208
2.10 Maintenance Considerations: Sediment Removal 216
2.10.1 Excavation 217
2.10.1.1 Hydraulic dredges 217
2.10.1.2 Conventional draglines 217
2.10.1.3 Crane-operated scrapers 219
2.10.1.4 Front-end loaders 219
2.10.1.5 Reducing excavation costs 219
2.10.2 Handling/Dewatering 222
2.10.3 Disposal 223
2.11 Particulars of the Alberta Foothills and Front 225
Ranges as they Relate to Sediment Pond Design

TABLE OF CONTENTS (CONTINUED)

Page

2.11.1 Topography 225
2.11.2 Winter Operations 227
2.11.2.1 Water temperature 227
2.11.2.2 Ice formation 228
2.11.2.3 Inflow conditions 229
2.11.2.4 Ambient air temperature 229
2.11.2.5 Designing ponds for water conditions 230
2.11.3 Overburden Characteristics 230
2.11.4 Wind 233

3. ALTERNATIVES TO SEDIMENT REMOVAL BY PONDING 234
3.1 Augmentative Technologies 234
3.1.1 Introduction 234
3.1.2 Land Modification 234
3.1.2.1 Effectiveness of land modification techniques ... 236
3.1.3 Mulches 238
3.1.3.1 Effectiveness of mulches 238
3.1.4 Vegetation 241
3.1.4.1 Effectiveness of vegetation 242
3.1.5 Diversion structures 243
3.1.5.1 Effectiveness of diversion structures 252
3.1.6 Check Dams 252
3.1.6.1 Effectiveness of check dams 254
3.1.7 Filter Fences 256
3.1.7.1 Effectiveness of filter fences 256
3.1.8 Vegetative Filter Strips 257
3.1.8.1 Effectiveness of vegetative filter strips 260
3.1.9 Sediment Traps 260
3.1.9.1 Effectiveness of sediment traps 262
3.1.10 Settling Pond Modifications 262

TABLE OF CONTENTS (CONTINUED)

Page

3.1.10.1 Effectiveness of settling pond modifications 277
3.1.11 In-Pond Chemical Treatment 277
3.1.11.1 Effectiveness of in-pond chemical treatment 277
3.1.12 Applicability to Alberta Foothills and Front Ranges 278
3.2 Alternative Technologies 279
3.2.1 Introduction 279
3.2.2 Chemical Treatment Plant 283
3.2.2.1 Description 283
3.2.2.2 Traditional applications 288
3.2.2.3 Advantages and disadvantages 288
3.2.2.4 Design and operational considerations 290
3.2.2.5 Suitability for sediment removal applications in the Alberta foothills and front ranges ... 293
3.2.3 Centrifugation 293
3.2.3.1 Description 293
3.2.3.2 Traditional applications 294
3.2.3.3 Advantages and disadvantages 298
3.2.3.4 Design and operational considerations 298
3.2.3.5 Suitability for sediment removal applications in the Alberta foothills and front ranges ... 300
3.2.4 Vacuum Filtration 301
3.2.4.1 Description 301
3.2.4.2 Traditional applications 304
3.2.4.3 Advantages and disadvantages 304
3.2.4.4 Design and operational considerations 304
3.2.4.5 Suitability for sediment removal applications in the Alberta foothills and front ranges ... 307
3.2.5 Hydrocyclones 309
3.2.5.1 Description 309

TABLE OF CONTENTS (CONTINUED)

Page

3.2.5.2 Traditional applications 309
3.2.5.3 Advantages and disadvantages 311
3.2.5.4 Design and operational considerations 311
3.2.5.5 Suitability for sediment removal applications ... in the Alberta foothills and front ranges 314
3.2.6 Ultrafiltration 314
3.2.6.1 Description 314
3.2.6.2 Traditional applications 320
3.2.6.3 Advantages and disadvantages 320
3.2.6.4 Design and operational considerations 321
3.2.6.5 Suitability for sediment removal applications ... in the Alberta foothills and front ranges 324
3.2.7 Cross Flow Microfiltration 325
3.2.7.1 Description 325
3.2.7.2 Traditional applications 328
3.2.7.3 Advantages and disadvantages 328
3.2.7.4 Design and operational considerations 330
3.2.7.5 Suitability for sediment removal applications ... in the Alberta foothills and front ranges 333
3.2.8 Electrokinetic Methods 334
3.2.8.1 Description 334
3.2.8.2 Traditional applications 335
3.2.8.3 Advantages and disadvantages 338
3.2.8.4 Design and operational considerations 338
3.2.8.5 Suitability for sediment removal applications ... in the Alberta foothills and front ranges 339
3.2.9 Cost Considerations 340

4. DISCUSSION 347
4.1 Settling Pond Design 347

TABLE OF CONTENTS (CONTINUED)

Page

4.1.1 Sedimentation Models 347
4.1.2 Current Alberta Practice 350
4.1.3 Designing Settling Ponds for Alberta Conditions . 351
4.2 Alternatives to Sediment Removal by Ponding 352
4.2.1 Augmentative Technologies 352
4.2.2 Alternative Technologies 352

5. REFERENCES CITED 358

LIST OF TABLES

Page

1. Waste water quality requirements of the Alberta Coal Mining Waste Water Effluent Guidelines 5

2. Runoff curve numbers (CN) for selected agricultural suburban, and urban land uses (antecedent moisture condition II) 10

3. Runoff curve numbers (CN) for hydrologic soil-cover complexes (antecedent moisture condition II) 12

4. Basic procedures for estimating a design flood hydrograph 16

5. Sediment storage volume requirements 20

6. Typical C_s factor values 26

7. Design criteria used for sizing detention volume in sediment ponds 39

8. Effects of varying particle size on effluent concentration with a constant detention time 51

9. Revised USEPA and OSM quality requirements for mine drainage 60

10. Dam hazard classification and return period storm for emergency spillway design 64

11. Summary of available pond models 86

LIST OF TABLES (CONTINUED)

Page

12. Ward's regression equations 91

13. Summary of DEPOSITS predicted trap efficiency 117
for field studies

14. Predicted and observed trap efficiencies for 118
DEPOSITS AND CSTRS models

15. Predicted and observed peak effluent TSS 119
concentrations for overflow rate, DEPOSITS,
and CSTRS models

16. Summary of predicted and observed results for 133
tests of the BASIN model

17. Summary of the Brune sedimentation model 139

18. Summary of the Churchill sedimentation model 140

19. Summary of steady state, quiescent flow, ... 141
overflow rate sedimentation models
(including USEPA model)

20. Summary of unsteady, quiescent flow, overflow 142
rate sedimentation models (including Tapp's methods)

21. Summary of unsteady, turbulent flow, overflow 143
rate sedimentation models

22. Summary of the DEPOSITS sedimentation model 144

LIST OF TABLES (CONTINUED)

Page

23. Summary of the CRTRS sedimentation model 146

24. Summary of the BASIN sedimentation model 147

25. Summary of the CREAMS sedimentation model 148

26. Description of the SEDIMOT II watershed model 151

27. Description of the CREAMS nonpoint source model 153

28. Description of the USEPA stormwater management 155
(SWMM) watershed model

29. Description of the KINEROS watershed model 157

30. Description of the USGS DR_3M and DR_2M - QUAL 159
watershed model

31. Design model cost evaluation; summary of input 163
data requirements for the Brune sedimentation model

32. Design model cost evaluation; summary of output 165
data from the Brune sedimentation model

33. Design model cost evaluation; summary of input 166
data requirements for the USEPA overflow rate model

34. Design model cost evaluation; summary of output 168
data from the USEPA overflow rate model

LIST OF TABLES (CONTINUED)

Page

35. Design model cost evaluation; summary of input data requirements for the DEPOSITS reactor model 169

36. Design model cost evaluation; summary of output data from the DEPOSITS reactor model 174

37. Design model cost evaluation; summary of input data requirements for the CSTRS reactor model 177

38. Design model cost evaluation; summary of output data from CSTRS reactor model 179

39. Design model cost evaluation; settling pond capital cost predicted by various sedimentation models 182

40. Mine B case history; computer model input data 200

41. Mine B case history; summary of inlet pond influent characteristics 202

42. Mine B case history; summary of inlet pond outlet characteristics 204

43. Mine B case history; summary of outlet pond outlet characteristics 206

44. Mine drainage dewatering methods and final densities 224

45. P factors for contouring 237

LIST OF TABLES (CONTINUED)

Page

46. Application of common mulch materials 239

47. Sediment traps, design specifications 264

48. Normal operating ranges for alternative sediment removal technologies 280

49. Rejected alternative technologies 284

50. Comparison of configurations of commercially available ultrafiltration membrane equipment 322

51. Alternative technologies cost evaluation; common wastewater characteristics 341

52. Alternative technology cost evaluation 342

53. Sensitivity of peak effluent concentrations predicted by the DEPOSITS model to variations in input parameters 348

54. Greatest rainfall in 24 hours at selected Alberta locations 353

55. Summary of alternative technologies 354

LIST OF FIGURES

Page

1. Schematic of sediment detention basin showing sediment .. storage, permanent pool, detention volume and flood storage volume (Barfield, Warner, and Haan 1981) 17

2. Iso-erodent map for Alberta (Tajek and Pettapiece 1984) . 23

3. Wischmeier nomograph (Haan and Barfield 1978) 24

4. Boyce curve for Da (Haan and Barfield 1978) 30

5. Renfro curve for Dc (Haan and Barfield 1978) 32

6. Pit detention time (Barfield, Warner, and Haan 1981) 33

7. Illustration of the method for determining the .... detention time for plug of flow 42

8. Illustration of the variables used in calculating .... detention storage time. 44

9. Triangular hydrograph assumptions and detention storage . time equations (Barfield, Warner, and Haan 1981) 46

10. Illustration of reservoir design for detention time 48

11. Illustration of the effects of permanent pool on plug ... flow detention time (Wilson, Barfield, and Moore 1982) 49

12. Controlled flow hydrographs 53

LIST OF FIGURES (CONTINUED)

Page

13. Settling pond hydrographs for controlled flow conditions. 55

14. Derivation sketch for settleable solids equations using . an Imhoff cone (Barfield, Warner, and Haan 1981) 61

15. Chart of sedimentation diameter versus fall velocity .... in quiescent water assuming a specific gravity of 2.65 and spherical shaped particles (Barfield, Warner, and Haan 1981) 67

16. Schematic of electric double layer theory (Adapted ...... from Tapp, Barfield, and Griffin 1981) 71

17. Illustration of forces of attraction and repulsion ...... near a colloidal particle (Adapted from Tapp, Barfield, and Griffin 1981) 72

18. Illustration of the processes involved in chemical ...... bridging (Adapted from Tapp, Barfield, and Griffin 1981) 73

19. Comparison of coagulant curves for several different .... coagulants (Singley 1972) 76

20. Potential for requiring flocculants for various .... minimum incoming particle size 77

21. Temperature profiles in Turtle Lake (Ford and Stefan .... 1980) 82

22. Illustration of mixing zones in a pond (Ford and Stefan . 1980) 83

LIST OF FIGURES (CONTINUED)

Page

23. Brune's trap efficiency curves (Barfield, Warner, and Haan 1981) ... 87

24. Churchill's curves for determining percent of incoming silt passing through reservoir as related to sedimentation index, TVA reservoirs (Barfield, Warner, and Haan 1981) .. 89

25. Illustration of sediment flow trajectories in an ideal rectangular sediment pond (Barfield, Warner, and Haan 1981) .. 94

26. Sediment trapping efficiency of a rectangular reservoir with steady state turbulent flow. (Barfield, Warner, and Haan 1981) . 100

27. A single continuous stirred reactor. (Wilson, Barfield, and Moore 1982) 104

28. A series of continuous stirred reactors. (Wilson, Barfield, and Moore 1982) 106

29. Schematic of a plug flow reactor. (Wilson, Barfield, and Moore 1982) ... 108

30. A single CSTR with short-circuiting and dead space (Wilson, Barfield, and Moore 1982) 110

31. Pond divided into a series of CSTRS. (Wilson, Barfield, and Moore 1982) 120

LIST OF FIGURES (CONTINUED)

Page

32. Pond run 18. (Wilson, Barfield, and Moore 1982) 125

33. Pond run 19. (Wilson, Barfield, and Moore 1982) 126

34. Pond run 22. (Wilson, Barfield, and Moore 1982) 127

35. Pond run 24. (Wilson, Barfield, and Moore 1982) 128

36. Pond run 26. (Wilson, Barfield, and Moore 1982) 129

37. Pond run 28. (Wilson, Barfield, and Moore 1982) 130

38. Schematic representation of a pond as used in the BASIN . Model (Wilson, Barfield, and Collivier 1984) 131

39. Case history mine A: pond B general arrangement 186

40. Case history mine A: natural watershed particle size ... distributions 189

41. Case history mine B: water management plan 194

42. Case history mine B: east sedimentation ponds 196

43. Case history mine B: pit dewatering arrangement 198

44. Case history mine B: model watershed dissection 199

45. Case history mine B: predicted inflow to inlet pond 209

46. Case history mine B: predicted inlet pond outflows 210

LIST OF FIGURES (CONTINUED)

Page

47. Case history mine B: predicted outlet pond outflows 211

48. Case history mine B: predicted inlet pond influent TSS . 212

49. Case history mine B: predicted inlet pond outflow TSS .. 213

50. Case history mine B: predicted outlet pond outflow TSS . 214

51. Case history mine B: predicted particle size distributions 215

52. Small hydraulic dredge (National Car Rental undated) 218

53. Typical crane-operated scraper arrangement (Thronson 1976) 220

54. High capacity crane-operated scraper arrangement (Thronson 1976) 221

55. Rainfall simulator for size distribution of eroded sediment using disturbed samples (Barfield, Warner, and Haan 1981) 232

56. Diversion dike construction details (USEPA 1976) 246

57. Diversion swale construction details (USEPA 1976) 247

58. Diversion channel construction details (USEPA 1976) 248

59. Grassed waterway construction details (USEPA 1976) 249

LIST OF FIGURES (CONTINUED)

Page

60. Grassed waterway with stone center construction details . (USEPA 1976) 250

61. Straw bale checkdam construction details (USEPA 1976) ... 255

62. Example outflow concentrations predicted by the GRASFIL . model (Hayes, Barfield, and Barnhisel 1982) 259

63. Schematic of model sediment trapping in a sediment ...... filter (Barfield and Hayes 1980) 261

64. Earth outlet sediment traps (USEPA 1976) 263

65. Circulations in sediment control ponds (adapted from .... Ward, Haan, and Tapp 1979) 266

66. Sedimentation pond flow patterns (Ward, Haan, and .... Tapp 1979) 268

67. Inlet baffle systems (Ward, Haan, and Tapp 1979) 269

68. Typical inlet baffle design (Ward, Haan, and Tapp 1979) . 270

69. Pond directional baffle systems (Ward, Haan, and .... Tapp 1979) 271

70. Probable flow patterns with a twin riser spillway .... (Ward, Haan, and Tapp 1979) 273

71. Typical siphon outlet (Taylor and Jenkins 1982) 275

LIST OF FIGURES (CONTINUED)

Page

72. Typical floating outlet (Taylor and Jenkins 1982) 276

73. Typical solids contact slurry recirculation clarifier ... (Graver Water 1982) 287

74. Typical chemical treatment design concept for runoff .... sediment removal applications 289

75. Cross section of a solid bowl centrifuge (Roe 1983) 295

76. Grade efficiency curves for some common solid-liquid .... separation devices (Svarovsky 1979a) 296

77. Effect of feed concentration and flowrate on cut size ... X_{50} for scroll-type centrifuge (Svarovsky 1979b) 297

78. Rotary drum vacuum filter (Svarovsky 1979a) 303

79. Representative curve for cake formation rate versus ..... feed solids concentration (Roe 1983) 306

80. Representative curve for cake formation rate versus ..... cycle time (Roe 1983) 308

81. Cross section of a hydrocyclone (Nawrocki and Pietrzak .. 1976) 310

82. Graph of pressure drop versus throughput capacity for ... two hydrocyclones (Roe 1983) 313

LIST OF FIGURES (CONTINUED)

Page

83. Schematic diagram of ultrafiltration process (Tang, ... Chu, and Boroughs 1980) 315

84. Tubular ultrafiltration module (Svarovsky 1979c) 317

85. Plate and frame ultrafiltration module (Svarovsky 1979c). 318

86. Cross flow microfiltration flow pattern (Neptune .... Microfloc 1983) 326

87. Schematic representation of a cross flow .... microfiltration unit (Schneider and Klein 1982) 327

88. Typical cross flow microfiltration module (Neptune ...... Microfloc 1983) 329

89. Cross flow microfiltration filtrate rate as a function .. of time (Schneider and Klein 1982) 332

90. The electrofiltration process (Dorr-Oliver 1984) 336

91. Scale model of the electrofilter (Freeman 1982) 337

ABSTRACT

Settling ponds are currently the most common structures used for the control of sediment contained in runoffs discharged from disturbed areas in Alberta. This document provides a critical evaluation of currently available settling pond design methodologies and examines some of the technologies that could be used as alternatives to sediment removal by ponding.

The pond design methodologies described herein vary greatly in their sophistication and in their ability to accurately predict pond performance. It was found that the more sophisticated models which consider the greatest number of relevant parameters exhibited the highest potential for reliable predictions of pond performance. However it was also determined that the quality of the outputs provided by these sophisticated models is very sensitive to the quality of the input data used. The use of inputs which are poorly correlated to site specific conditions can result in predictions of pond performance that bear little or no resemblance to reality.

The literature review also identified a number of augmentative and alternative technologies that can be used to either maximize the performance of a particular pond size or to replace the pond altogether. Augmentative technologies reduce the sediment load on a pond or improve the efficiency of the physical processes by which sediment is retained in a pond. Alternative technologies can be used in place of settling ponds in applications for which impoundments require too much space or are incapable of providing the desired effluent quality. Many of the alternative technologies identified are capable of maintaining a very high effluent quality but usually at a relatively high cost.

1. INTRODUCTION

1.1 NATURE AND SCOPE OF THE STUDY

There is increasing concern regarding the environmental implications of land-disturbing activities in western Canada. Untreated runoff from surface mining operations, sand and gravel pits, and large construction sites has the potential to contaminate receiving surface waters with high sediment loads. The study of sedimentation shows that the process involves soil erosion by water, transportation (primarily by flowing water) and eventual deposition of at least the larger particles of the sediment. The traditional solution to the problem of removing the sediment has been to construct sedimentation basins in which the runoff is retained for a period sufficient to allow suspended particulates to settle to the bottom of the pond. Over the years, various researchers and regulatory authorities have advocated the use of a number of different sediment pond design approaches of varying complexity and sophistication.

The use of sediment ponds does not influence the erosion and transportation processes and therefore the ponds can be viewed only as an 'end-of-the-pipe' treatment technology. Recent trends toward effluent quality requirements have prompted researchers and designers to seek economical alternatives that influence the entire spectrum of the process of sedimentation (i.e., erosion, transportation, and deposition).

The overall objective of this study is to critically evaluate methods or technologies that may help achieve effective control over erosion, transportation, and eventual deposition of sediment. The end goal is to achieve compliance with regulatory requirements.

1.2 TERMS OF REFERENCE

The scope of work for this report is based on Terms of Reference issued by Dr. Paul F. Ziemkiewicz, Director, Reclamation Research, Alberta Energy and Natural Resources on 1984 June 04. The scope includes:

1. A critical review of settling pond design literature;
2. An evaluation of possible alternatives to sediment removal by ponding;
3. The preparation of a catalogue of design procedures that will characterize each method's strengths and weaknesses; and
4. A commentary on the suitability of the methods with respect to environmental conditions found in the Alberta foothills and front ranges.

1.3 LITERATURE SEARCH

Because of the scope and the nature of the study, literature searches to gather pertinent data assumed great importance. Monenco carried out literature searches in three phases:

1. In-house data search and computerized search utilizing available environmental and engineering data bases;
2. Manual searches through published environmental and engineering data bases not accessible through the computer data bases; and
3. Sub-contract to Mr. B.J. Barfield, Professor at the University of Kentucky, Lexington, Kentucky, a recognized authority in North America on problems associated with sedimentation.

Computerized searches were conducted through Monenco's in-house computer system. Manual searches utilized Monenco's interlibrary loan services, the University of Calgary, the City of Calgary Public Library, and the University of British Columbia. Searches were conducted by subject, keywords, associated subject keywords, and names of principal authors working in the field of the study.

Unfortunately, the literature search did not uncover a large volume of Canadian literature describing settling pond design methodologies that were not based on American references. The search quickly revealed that much of the current research on computer sedimentology models originates at the University of Kentucky in in Lexington, Kentucky. It was concluded therefore, that it would be desirable to emphasize the results of University of Kentucky research and, to that end, Monenco solicited the services of Mr. B.J. Barfield of the University of Kentucky's Agricultural Engineering Department.

2. SETTLING POND DESIGN

2.1 INTRODUCTION

Settling ponds are the most common off-site control measures used to reduce the discharge of sediment from disturbed lands to downstream surface water bodies. This section will examine some of the fundamental operating principles associated with settling ponds and describe the various procedures that have been used to design sediment removal impoundments. The application of specific design methodologies to some currently operating ponds will be presented along with a commentary on how the particulars of watersheds in the Alberta foothills and front ranges restrict the suitability of some design models for application to this area.

2.2 REGULATORY OBJECTIVES

In Alberta some of the major land-disturbing activities are those associated with surface coal mining. The requirements of the 'Alberta Coal Mining Waste Water Effluent Guidelines', summarized in Table 1, are typical of those promulgated by many authorities in that water discharged from a disturbed area is required to meet a specific suspended solids limitation for all flows less than those associated with a specified storm event (usually the 1 in 10 year, 24-hour storm).

These kinds of requirements are established to minimize the negative impact that elevated suspended solids levels can have on a watercourse. Suspended solids can increase fish mortality by clogging gills and disturb fish breeding by destroying aquatic habitats. High suspended solids concentations can also reduce the recreational value of surface water and increase the cost of water use by downstream consumers (Alberta Environment 1977).

Table 1. Waste water quality requirements of the Alberta Coal Mining Waste Water Effluent Guidelines

Water Contaminant or Parameter	Standard 24-hour Average
Total Suspended Solids	50 milligrams/litre Max. Absolute or 10 milligrams/litre Max. above the natural background concentration, whichever is the greater.
Total Iron	3.5 milligrams/litre Max. Absolute.
pH	Within the range 6.5 - 9.5.

Note: Standards contained above shall be null and void in the event of a 10-year storm being exceeded on the mine site.

2.3 DEVELOPING DESIGN STORMS

2.3.1 Introduction

A critical input for many of the settling pond design methodologies discussed in subsequent sections is the design flood hydrograph. In Alberta, the design flood hydrograph is defined as the runoff resulting from a 1 in 10 year return period storm with a 24-hour duration. It is important to realize that this event is not the 1 in 10 year return period flood, as it is often rather loosely termed. The flood hydrograph so defined will have a return period that is a function of the initial conditions of the catchment at the onset of the storm. For example, a 1 in 10 year storm combined with very dry initial conditions may lead to a 1 in 2 year flood and conversely may lead to a 1 in 50 year flood if the initial conditions are very wet.

There are many different ways to estimate the design flood hydrograph. In general, reasonable results may be obtained provided that site-specific data are used and whenever possible this should be done. The hydrologic processes involved in converting precipitation to streamflow are very complex and it is usually not practical to even attempt to take them into account fully when estimating the design flood hydrograph. The procedure usually adopted is as follows:

1. Estimation of the design storm in time increments appropriate for the size of the catchment under consideration;
2. Estimation of the rainfall losses that translate the storm rainfall to effective rainfall;
3. Estimation of the unit hydrograph of the catchment; and
4. Computation of the design flood hydrograph by combining the above factors. This procedure will include estimation of the baseflow contribution to the hydrograph.

The above factors are discussed in some detail in the following sections. Additional information on the estimation of runoff volumes and frequencies for disturbed regions of Alberta's eastern slopes can be found in Ziemkiewicz (1983).

2.3.2 Design Storm

The design storm is usually estimated from data collected at a rainfall measuring station close enough to the catchment of interest to be considered representative of conditions in the catchment. Provided that several years data have been collected at the station, an intensity-duration-frequency (IDF) curve can be constructed. In Canada, if the station is operated by Atmospheric Environment Services, they will calculate the IDF curve for a nominal fee.

The IDF curve may then be used to determine the depths of rainfall for different storm durations and frequencies of occurence. For settling pond design in Alberta the duration is usually set at 24 hours and the frequency at 1 in 10 years. In order to use the rainfall, it is usually necessary to divide the storm up into smaller time increments such as one hour. The actual size of the time increment is a function of the response time of the catchment. If one assumes that the 24 hour storm is comprised of shorter durations with the same frequency (i.e. 1 in 10 years in Alberta), then the IDF curve may be used to calculate the rainfall depth increments. These rainfall increments must then be rearranged to form a storm profile reasonable for the general area in which the catchment of interest is located. We know of no studies of Alberta rain storms that assist this rearrangement procedure. Several alternatives exist, an example of which is the SCS method (U.S. Soil Conservation Service 1973) which avoids the need to calculate the increments from the IDF curve. The SCS method has two storm patterns called Type I and Type II. Type I is applicable to Hawaii, Alaska, and the coastal side of the Sierra Nevada and the Cascade Mountains. Type II is applicable to the remainder of the United States, Puerto Rico and the Virgin Islands. If one is applying the SCS method to Alberta conditions, the Type II pattern should be used. Given the total rainfall for the 24-hour storm, the Type II curve distributes the rainfall into any desired time increments.

2.3.3 Estimation of Rainfall Losses

During a rain storm event rainfall losses are comprised mainly of depression storage and infiltration. These are often termed abstractions from rainfall. Surface storage is the volume of water required to fill depressions and other storages before surface runoff begins. Ultimately this water either infiltrates or is evaporated.

In large rainfall events the major rainfall abstraction is infiltration. In general, infiltration is a function of soil physical properties, vegetative cover, antecedent soil water conditions, rainfall intensity, and the slope of the infiltrating surface.

Both surface storage and infiltration are extremely heterogeneous within a catchment and mathematical models of these processes use indices representing average values over the catchment of interest. Models of these processes have been developed and some have received wide usage (e.g. surface storage (Linsley, Kohler, and Paulhus 1958) and infiltration (Horton 1940; Philip 1954)). A widely used technique developed by the US Soil Conservation Service (1972) combines the surface storage and infiltration losses. The accumulated runoff volume is defined as,

$$Q = \frac{(P-0.2S)^2}{P + 0.8S} \qquad P > 0.2S \tag{1}$$

where, Q is the accumulated runoff volume (rainfall excess) in inches, P is the accumulated precipitation in inches, and S is a parameter defined by,

$$S = \frac{1000}{CN} - 10 \tag{2}$$

where CN is known as the curve number. The curve number is a function of both the soil type and the antecedent conditions of the catchment. The curve number is the only parameter of the model that requires estimating and, since it attempts to account for both depression storage and infiltration, the success of the method is very much dependent on the selection of the curve number. Typical CN numbers from two different references are presented in tables 2 and 3. The curve number must be selected to reflect the conditions of the catchment including mining-related activities such as pre-stripping and reclamation. An area-weighted average curve number may be calculated for the catchment to achieve this.

It is important to recognize the great sensitivity of the hydrograph shape to variations in the runoff curve number. Unless CN numbers have been calibrated to the specific watershed in question, errors in peak flow predications in the order of 50% are not unusual (see Table 53 for illustration of sensitivity). Consequently, peak storm flow and volume estimates based on uncalibrated CN numbers must be used with considerable caution.

2.3.4 Unit Hydrograph Estimation

Sections 2.3.2 and 2.3.3 summarized how a design storm may be developed and how to estimate the rainfall losses to arrive at the increments of effective or net rainfall (rainfall minus losses). By far the most common technique for converting the effective rainfall into streamflow is the unit hydrograph method. A unit hydrograph of duration D hours is the runoff caused by a unit effective rainfall input occurring over a duration of D hours. Unit hydrograph theory is based upon the premise of a linear system (i.e., 2 cm of effective rainfall occurring in D hours produces exactly twice the runoff hydrograph of 1 cm of effective rainfall in D hours). If reasonable account has been taken of the depression storage and infiltration, this is a reasonable assumption.

The shape of a unit hydrograph is a function of many variables including the rainfall duration D, channel slope, drainage

Table 2. Runoff curve numbers (CN) for selected agricultural, suburban, and urban land uses (antecedent moisture condition II).[a]

Land Use Description		Hydrologic Soil Group			
		A[g]	B[h]	C[i]	D[j]
Cultivated land[b]: without conservation treatment		72	81	88	91
: with conservation treatment		62	71	78	81
Pasture or range land: poor condition		68	79	86	89
good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or forest land: thin stand, poor cover, no mulch		45	66	77	83
good cover[c]		25	55	70	77
Open spaces, lawns, parks, golf courses, etc.					
good condition: grass cover on 75% or more of the area		39	61	74	80
fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious).		81	88	91	93
Residential[d]:					
Average lot size	Average % Impervious[e]:				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84

continued....

Table 2. Concluded.

Paved parking lots, roofs, driveways, etc.[f]	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers[f]:	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

[a] Source: Barfield, Warner, and Haan (1981).

[b] For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

[c] Good cover is protected from grazing and litter and brush cover soil.

[d] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[e] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[f] In some warmer climates a curve number of 95 may be used.

[g] These soils have a high infiltration rate. They are chiefly deep, well-drained sands or gravels (Low runoff potential).

[h] These soils have a moderate infiltration rate when thoroughly wet. They are chiefly moderately deep, well-drained soils of moderately fine to moderately course texture.

[i] These soils have a slow infiltration rate when wet. They are chiefly moderately deep, well-drained soils of moderately fine to moderately course texture.

[j] These soils have a very slow infiltration rate. They are chiefly clay soils with a high swelling potential, soils with a permanently high water table, soils with a clay pan at or near the surface and shallow soils over nearly impervious materials. (High runoff potential).

Table 3. Runoff curve numbers (CN) for hydrologic soil-cover complexes (antecedent moisture condition II).[a]

Land Use or Cover	Treatment or Practice	Hydrologic Condition for Infiltrating	Hydrologic Soil Group[g]			
			A	B	C	D
Fallow	SR[b]		77	86	91	94
Row crops	SR	Poor	72	81	88	91
	SR	Good	67	78	85	89
	C[c]	Poor	70	79	84	88
	C	Good	65	75	82	86
	C&T[d]	Poor	66	74	80	82
	C&T	Good	62	71	78	81
Small grain	SR	Poor	65	76	84	88
	SR	Good	63	75	83	87
	C	Poor	63	74	82	85
	C	Good	61	73	81	84
	C&T	Poor	61	72	79	82
	C&T	Good	59	70	78	81
Close-seeded legumes[e] or rotation meadow	SR	Poor	66	77	85	89
	SR	Good	58	72	81	85
	C	Poor	64	75	83	85
	C	Good	55	69	78	83
	C&T	Poor	63	73	80	83
	C&T	Good	51	67	76	80

continued....

Table 3. Concluded.

Land Use or Cover	Treatment or Practice	Hydrologic Condition for Infiltrating	Hydrologic Soil Group[g]			
			A	B	C	D
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	C	Poor	47	67	81	88
	C	Fair	25	59	75	83
	C	Good	6	35	70	79
Meadow (permanent)			30	58	71	78
Woods (farm woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads (dirt)[f]			72	82	87	89
(hard surface)[f]			74	84	90	92

[a] Source: U.S. Department of the Interior (1977).

[b] SR = straight row

[c] C = contoured

[d] T = terraced

[e] Close-drilled or broadcast

[f] Including right-of-way

[g] Hydrologic soil groups are described in Table 2.

area and shape, number of small tributaries to the main channel, and the presence of lakes or manmade disturbances such as towns. There are many ways of estimating the unit hydrograph for a catchment in the absence of recorded data. Only one of the methods is discussed here. The reader is referred to any standard hydrology textbook for other methods.

The SCS method (Soil Conservation Service 1972) is based on a dimensionless unit hydrograph. Two parameters must be estimated to combine with the dimensionless unit hydrograph to yield the desired unit hydrograph. These parameters are the time to peak, Tp, and the peak flow, Qp. Tp is the time from the start of the effective rainfall to the peak flow of the unit hydrograph and Qp is the peak flow of the unit hydrograph. Tp is computed by

$$Tp = \frac{D}{2} + t_l \tag{3}$$

where D is the duration, in hours, of the effective rainfall, and t_l is the basin lag in hours. The basin lag time may be estimated in a number of ways. One method uses the relationship

$$t_l = 0.6t_c \tag{4}$$

in which t_c is estimated using Kirpich's (1940) formula

$$t_c = 0.00013 \ L^{0.77} \ (L/H)^{0.385}$$

where t_c is the catchment time of concentration in hours, L is the maximum length of flow in feet, and H is the difference in elevation in feet between the outlet of the catchment and the hydraulically most remote point in the catchment. When t_l has been estimated, it is usual to select D to be in the range of 0.1 to 0.25 of t_l. The selection of D also establishes the time increments into which the design storm must be broken.

2.3.5 Computation of the Design Flood Hydrograph

The preceding three sections have discussed most of the analytical procedures required to make an estimate of the design flood hydrograph. These procedures are summarized in Table 4. The only item not discussed is the baseflow contribution to the flood hydrograph. However, for the small catchments normally associated with settling ponds, groundwater input to the design flood hydrograph may usually be ignored.

2.3.6 Use of Local Data

It cannot be overemphasized that whenever possible, use should be made of data in or near the catchment of interest. Rainfall and runoff records for similar catchments near the area of interest may be used to make better estimates of rainfall losses and the unit hydrograph.

2.4 DESIGN REQUIREMENTS

The ultimate objective of any settling pond design exercise is to provide an estimate of the minimum total pond storage volume that will satisfy the applicable regulatory requirements. The total storage volume consists of the four major components shown in Figure 1.

2.4.1 Sediment Storage Volume (V_S)

The sediment storage volume is provided to store sediment trapped over the life of the structure, or between clean-outs. Sizing of this volume is typically based on the long-term sediment yield as opposed to that in a single storm.

2.4.2 Permanent Pool Volume (V_p)

The permanent pool is that volume of water above the sediment storage volume and below the crest of the emergency spillway

Table 4. Basic procedures for estimating a design flood hydrograph.

1. Determine design conditions (e.g., frequency and duration of design storm in Alberta are 1 in 10 years and 24 hours respectively).

2. Derive the IDF curve for the catchment using data from a nearby meteorological station.

3. Estimate the unit hydrograph for the catchment.

4. Determine the breakdown of the total storm input in D-hour increments.

5. Estimate the effective rainfall in each D-hour increment by subtracting the depression storage and infiltration from the rainfall.

6. Apply the estimated unit hydrograph to the increments of effective rainfall to determine the design flood hydrograph.

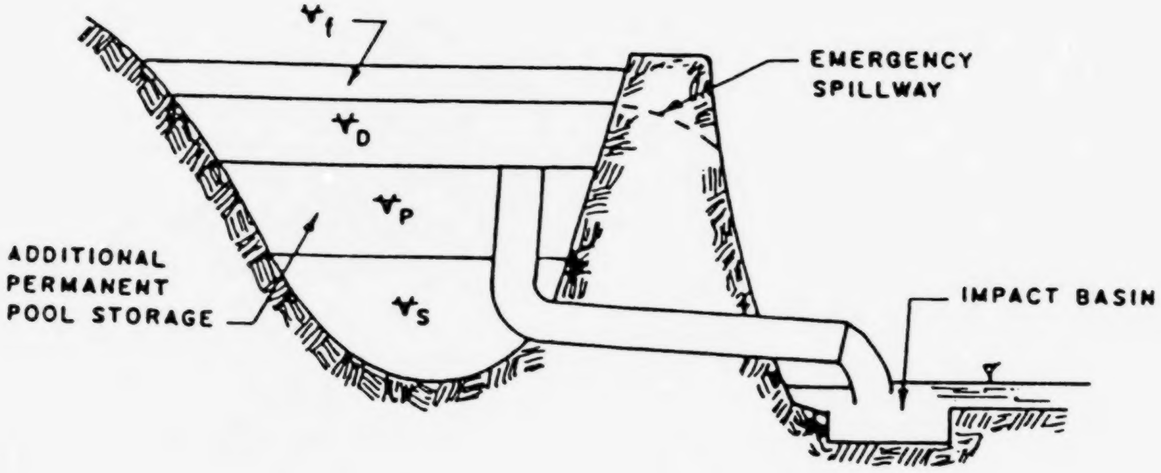

∀f
∀D
∀P
∀S
EMERGENCY SPILLWAY
ADDITIONAL PERMANENT POOL STORAGE
IMPACT BASIN

Figure 1. Schematic of sediment detention basin showing sediment storage, permanent pool, detention volume and flood storage volume. (Barfield, Warner and Haan 1981)

that is not dewatered after a storm. Reservoirs that do not have a permanent pool are said to be dewatered structures.

2.4.3 Detention Volume for Settling (V_d)

The detention volume for settling is that volume added to the pond to allow the stormwater to be detained long enough for the desired sediment fraction to settle out. This volume along with the permanent pool determines the performance of a pond for a given set of site-specific input conditions.

2.4.4 Flood Storage Volume

The flood storage is added above the detention volume to ensure that the reservoir does not overtop during a rare storm event. The return period of the event is established as a function of the downstream damage that would result if the impoundment was breached.

2.5 DESIGN APPROACHES

The design of a settling pond is completed by using one of a number of available methods to estimate the size and configuration of each of the previously described volumes. The following discussion outlines some of the basic design approaches that can be used to predict these volumes. A complete discussion of the design models that are used in some of the approaches for predicting sediment removal efficiency is presented in Section 2.6.

2.5.1 Sediment Storage Volume

Sediment storage volume calculations are based on sediment yields expected over the period of significant sediment discharge into the structure. These yields are translated into volumes by using data on sediment density as related to composition and reservoir operation. If periodic cleanout is used, the yield is calculated over the period between expected cleanouts. The sediment storage volume is typically not assumed to be an active portion of the reservoir during a runoff event since it will be filled in a worst case analysis.

Sediment yield is typically determined using one of four methods:

1. 'Rule-of-thumb' approach;
2. Gross erosion with sediment delivery ratio;
3. Sediment yield model; and
4. Reservoir survey method.

2.5.1.1 Rule-of-thumb approach. The most straightforward method of sizing the sediment storage volume involves the use of a rule-of-thumb estimate for sediment yields from watersheds in a particular area. This approach defines the required sediment storage volume simply as a function of the drainage area affected and the expected frequency of sediment removal. Some examples of the sediment yield estimates used in various jurisdictions in the United States are shown in Table 5.

Rule-of-thumb approaches provide only crude estimates of expected sediment yields because site-specific factors are not considered. The type and extent of land disturbing activity as well as the particulars of the local overburden and weather patterns can have a profound impact on the sediment yield from a specific watershed.

2.5.1.2 Gross erosion with sediment delivery ratio. This approach estimates the required sediment storage volume by using empirical methods to predict the gross erosion and delivery ratio for a particular watershed. It represents an improvement over the rule-of-thumb approach because site-specific watershed factors are considered. The annual volume of sediment delivered to an impoundment is estimated by multiplying the gross erosion per unit area by the delivery ratio and the total watershed area. Models for estimating gross erosion are well established but delivery ratio determinations are more art than science in many cases.

2.5.1.2.1 Gross erosion. The most commonly used model for calculating gross erosion is the Universal Soil Loss Equation (USLE)

Table 5. Sediment storage volume requirements.[a]

Authority	Requirement
Maryland	30 mm/ha drained
	13 mm/ha drained[b]
Kentucky	600 m^3/ha disturbed
West Virginia	380 m^3/ha disturbed[c]
USEPA	300 m^3/ha disturbed[c]

[a] Adapted from original table in USEPA (1976).

[b] To be cleaned when storage capacity drops below 13 mm/ha.

[c] To be cleaned when sediment accumulation approaches 60 percent of design sediment storage volume.

(Wischmeier and Smith 1965, cited by Haan and Barfield 1978), which was originally developed to estimate the average annual soil loss from agricultural areas. Because of its simplicity it has been widely used to evaluate the effects of different conservation plans on watershed erosion. The USLE is an empirical equation based on over 8000 plot years of data (in 1960) (Wilson, Barfield, and Moore 1982) and is given as

$$Y = R\ K\ LS\ C\ P \tag{6}$$

where Y is the average annual soil loss (mass/area), R is a rainfall erosion factor (usually expressed as the product of rainfall energy times the maximum thirty-minute intensity for a given rainstorm), K is a soil erodibility factor (mass/area - R unit), LS is a combined plot length and slope factor (dimensionless), C is a cropping management factor (dimensionless), and P is an erosion control practice factor (dimensionless).

The rainfall factor, R, accounts for the potential of rainfall to erode soil. Wischmeier (1959, cited by Haan and Barfield 1978) evaluated the relationship between 10 indicators of the force and energy of rainfall and what is known as rainfall erosivity and concluded that the best estimate of R was obtained from the product of rainfall energy and the maximum 30-minute intensity of rainfall. The resulting equation for R is

$$R = \frac{E\ I_{30}}{100} \tag{7}$$

where E is rainfall energy and I_{30} is the maximum 30-minute intensity of rainfall. Wischmeier also developed an empirical relationship for rainfall energy on a storm basis

$$E = 916 + 331 \log_{10} I \tag{8}$$

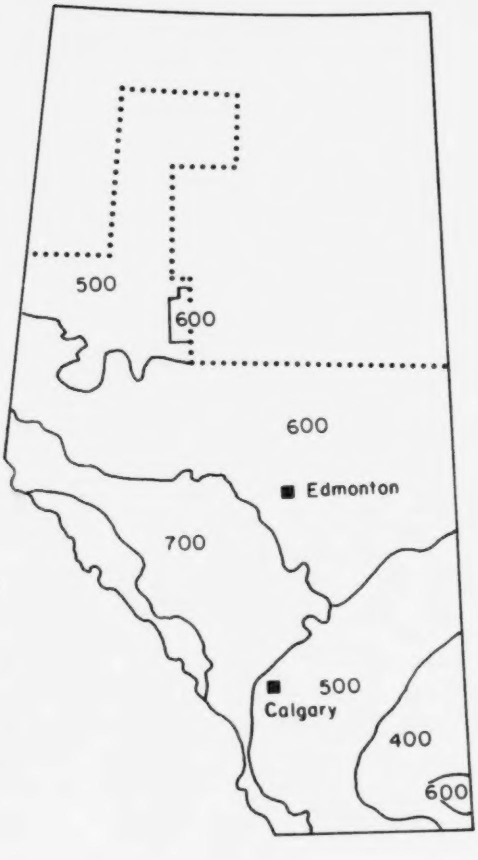
where I is the average intensity of the storm (note that this equation is presented in Imperial units with I in inches/hour). Hudson (1971, cited by Haan and Barfield 1978) found that this relationship works reasonably well for temperate climates like Alberta's. However, more precise estimates for R that accurately reflect conditions in Alberta, particularly those associated with prolonged periods of snow cover, are difficult to find in the literature. The iso-erodent map (i.e., map showing lines of equally erosive rainfall) shown in Figure 2 was recently prepared by Tajek and Pettapiece (1984) and probably represents the best information currently available on R values that can be used to predict average annual soil loss in Alberta.

The soil erodibility factor, K, accounts for the soil's resistance to erosion as well as its ability to take up water (infiltration capacity). These characteristics change with depth, making the erodibility dependent upon the soil horizon exposed and the development of a universally applicable predictive equation for K very difficult. Wischmeier, Johnson, and Cross (1971, cited by Haan and Barfield 1978) developed a nomograph that estimates K as a function of soil textural parameters, organic content, soil structure, and permeability. While this nomograph, which is shown in Figure 3, has limitations for applications involving clay subsoils and exposed spoil material, it is often the only usable tool the designer has at his disposal. A complete illustration of the use of the Wischmeier et al nomograph can be found in Barfield, Warner, and Haan (1981).

The length slope factor, LS, is used to evaluate the effects of plot length and slope on erosion. Wischmeier and Smith (1965, cited by Haan and Barfield 1978) proposed that the LS factor could be given by

$$LS = \left[\frac{L}{22.1}\right]^m \left[\frac{430\ SX^2 + 30\ SX + 0.43}{6.613}\right] \tag{9}$$

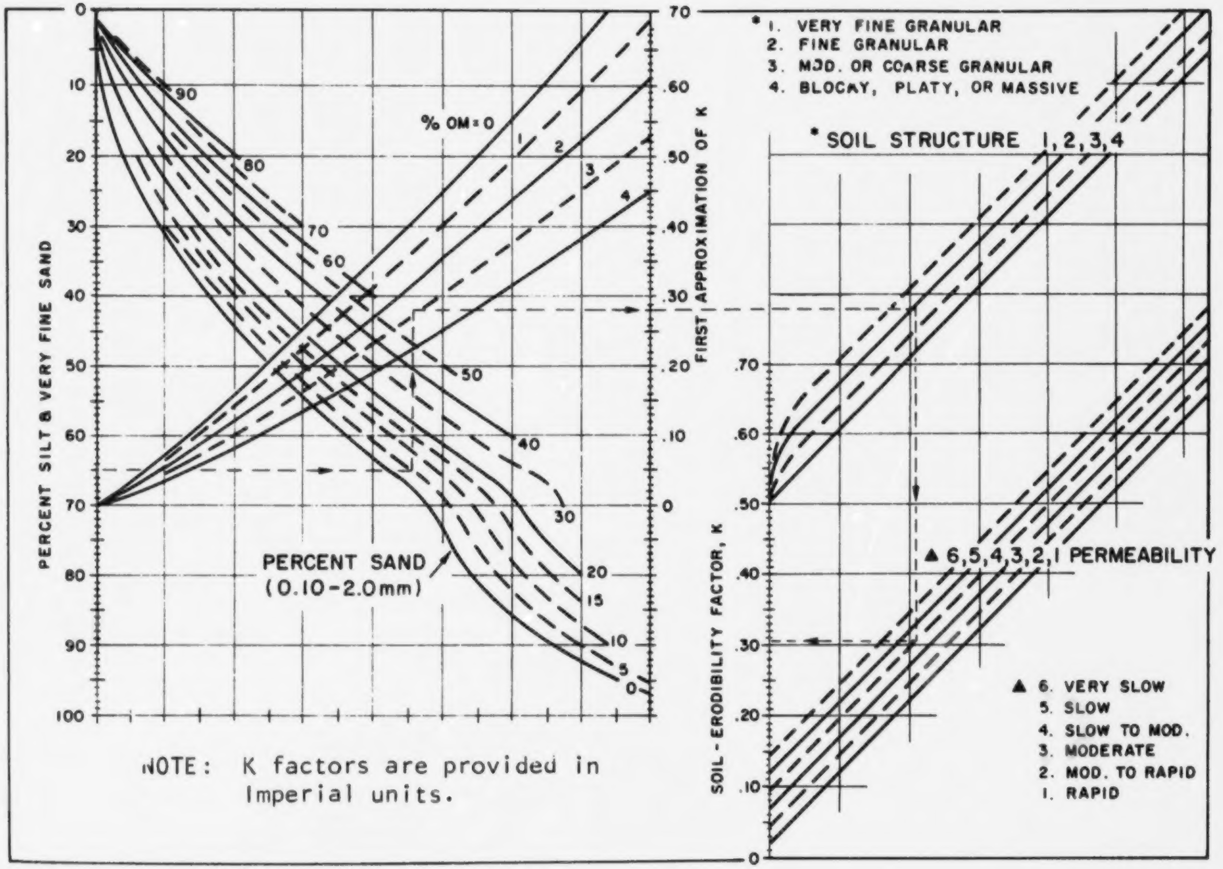
where L is slope length in metres, SX is the tangent of the slope angle, and m is a slope-dependent exponent. The slope length in


500
600
600
Edmonton
700
500
Calgary
400
600

R_T Factor notes:

1. R_T is a combined rainfall and winter erosion index to be used in place of R in the USLE to calculate average annual soil loss in a particular area.
2. Units of R_T are $\frac{MJ\cdot mm}{ha\cdot h}$

Figure 2. Iso-erodent map for Alberta. (Tajek and Pettapiece 1984)



NOTE: K factors are provided in Imperial units.

PROCEDURE: With appropriate data, enter scale at left and proceed to points representing the soil's % sand (0.10 to 2.0 mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: st + vfs 65%, sand 5%, OM 2.9%, structure 2, permeability 4. Solution: K = 0.31.

Figure 3. Wischmeier nomograph. (Haan and Barfield 1978)

equation 9 is defined as the distance from the point of origin of overland flow to the point that the slope decreases such that deposition occurs or until the flow enters a defined channel. The exponent m is related to slope by

Slope $\leq$ 3% $\quad m = 0.3$
Slope = 4% $\quad m = 0.4$
Slope $\geq$ 5% $\quad m = 0.5$

Equation 9 was based on data for slopes between 3 and 20% over lengths of up to 120 m and actually represents the ratio of soil loss at any length and slope relative to that at 22.1 m and 9%.

The cropping management factor, C, was originally defined (Wischmeier and Smith 1965, cited by Haan and Barfield 1978) as the ratio between soil loss of a land cropped under a specified condition to the soil loss of continuous fallow. The erosion control practice factor, P, was used to account for conservation practices such as strip cropping and contour forming. Barfield, Warner, and Haan (1981) proposed that the two factors be combined into a control practice factor, CP, which could be estimated according to

$$CP = C_s \cdot C_r \cdot C_o \tag{10}$$

where C_s is a factor accounting for surface stabilizing or protection treatments, C_r is a factor accounting for runoff reduction practices such as diversion berms, intercepter dikes, benches and terraces, and sectional drawdowns, and C_o is a factor accounting for any other erosion control practices that may be used. The overall CP would be equal to unity for a freshly denuded site with no conservation practices.

The C_s factor estimates the effect of various means of stabilizing exposed material on total sediment yield. Typical C_s values for a number of soil conditions and amendments are shown in Table 6. The C_r factor accounts for the fact that anything that slows

Table 6. Typical C_s factor values[a].

Condition	C_s Factor
1. Bare soil conditions	
freshly disked to 150 - 200 mm	1.00
after one rain	0.89
loose to 300 mm smooth	0.90
loose to 300 mm rough	0.80
compacted bulldozer scraped up and down	1.30
same except root raked	1.20
compacted bulldozer scraped across slope	1.20
same except root raked across	0.90
rough irregular tracked all directions	0.90
seed and fertilize, fresh,unprepared seedbed	0.64
same after six months	0.54
seed, fertilize and 12 months chemical	0.38
not tilled algae crusted	0.01
compacted fill	1.24-1.71
undisturbed except scraped	0.66-1.30
scarified only	0.76-1.31
sawdust 50 mm deep, disked in	0.61
2. Asphalt emulsion	
11700 L/ha	0.02
11300 L/ha	0.01-0.019
5700 L/ha	0.14-0.57
2800 L/ha	0.28-0.60
1400 L/ha	0.65-0.70
3. Dust binder	
5700 L/ha	1.05
11300 L/ha	0.29-0.78

continued...

Table 6. Concluded.

Condition	C_s Factor
4. Other chemicals	
450 Kg fiber glass roving with 560 - 1400 L/ha	0.01-0.05
Aquatain	0.68
Aerospray 70, 10 percent cover	0.94
Curasol AE	0.30-0.48
Petroset SB	0.40-0.66
PVA	0.71-0.90
Terra-Tack	0.66
wood fiber slurry, 1120 Kg/ha fresh	0.05
wood fiber slurry, 1600 Kg/ha fresh	0.01-0.02
wood fiber slurry, 3900 Kg/ha fresh	0.10
4. Seedings[b]	
temporary, 0 to 60 days[c]	0.40
temporary, after 60 days	0.05
permanent, 0 to 60 days[c]	0.40
permanent, 2 to 12 months	0.05
permanent, after 12 months	0.01
6. Brush	0.35
7. Excelsior blanket with plastic net	0.04-0.10

[a] Source: Haan and Barfield (1978)

[b] If plantings are used with mulches, use the minimum C_s values.

[c] If dry weather occurs at planting and emergence is a problem, extend the 0-60 days to a period when rainfall normally occurs.

runoff down or diverts it away from downslope areas reduces erosion. Chen (1974, cited by Haan and Barfield 1978) proposed that the effect of such diversions could be estimated by

$$C_r = 1/(N+1)^{1/2} \qquad (11)$$

where N is the number of diversions placed across a uniform slope.

It should be recognized that the USLE does have some significant limitations. It does not account for deposition and therefore will overpredict erosion rates from regions for which a significant potential for deposition exists. At present, the procedures available for predicting strip mine spoil erodiblities for use in the USLE are weak. Modifications to the equation have been proposed by various researchers, particularly in the rainfall factor, but improvements in prediction accuracy have been marginal (Onstad and Foster 1975). However, the above limitations notwithstanding, the USLE continues to provide a relatively straightforward methodology that provides simple, conservative estimates of gross erosion.

2.5.1.2.2 Delivery ratio. The USLE calculates gross erosion by assuming that no sediment is deposited as the runoff is discharged from a watershed. In fact, the sediment will have considerable opportunity for desposition between the upper reaches of a watershed and the inlet to a settling pond. In order to quantify the final sediment yield, a sediment delivery ratio is used. The sediment delivery ratio is defined (Haan and Barfield 1978) as

$$D = Y/A \qquad (12)$$

where Y is the sediment yield from a watershed and A is the gross erosion occurring on the watershed.

Accurate prediction of delivery ratios is more difficult than predicting gross erosion due to the lack of a large data base. Boyce (1975, cited by Haan and Barfield 1978) proposed that delivery ratio could be predicted based on the watershed area and summarized the data available. The results show that delivery ratio decreases as a power function of area ranging from near 1.0 for a drainage area of 2.6 ha to 0.1 for a drainage area of 20 000 ha. The data are so scattered about the line of best fit that one cannot place strong confidence in the estimates. Barfield, Warner, and Haan (1981) suggested that the delivery ratio from surface mined watersheds could be estimated by using,

$$DR = D_a \cdot D_v \cdot D_c \cdot D_p \qquad (13)$$

where DR is the delivery ratio and D_a, D_v, D_c, and D_p account for the effects of area, vegetation, channelization, and pit deposition respectively, on the delivery ratio.

D_a in equation 13 evaluates the area effects on sediment delivery ratio and can best be estimated using a site specific version of the Boyce curve shown in Figure 4. If a site specific version of this figure is not available and cannot be prepared easily, Figure 4 can be used to estimate D_a if its predicted value is used with considerable caution.

D_v accounts for the effect of vegetative sediment trapping on delivery ratio and can be estimated using (Holberger and Truett 1976, cited by Haan and Barfield 1978),

$$D_v = (L/0.305)^{-0.22} \qquad (14)$$

where L is the length of the flow path through the vegetated area in metres. Haan and Barfield (1978) suggested that the uncertainty associated with D_a values taken from Figure 4 and D_v values calculated using equation 14 warranted the use of some conservative guidelines for establishing the product of these two factors. They suggested that if the flow path is through normal vegetation the product of D_a and D_v should be calculated by taking the lower of the ordinate of Figure 4 or the result of equation 14 and setting the remaining factor

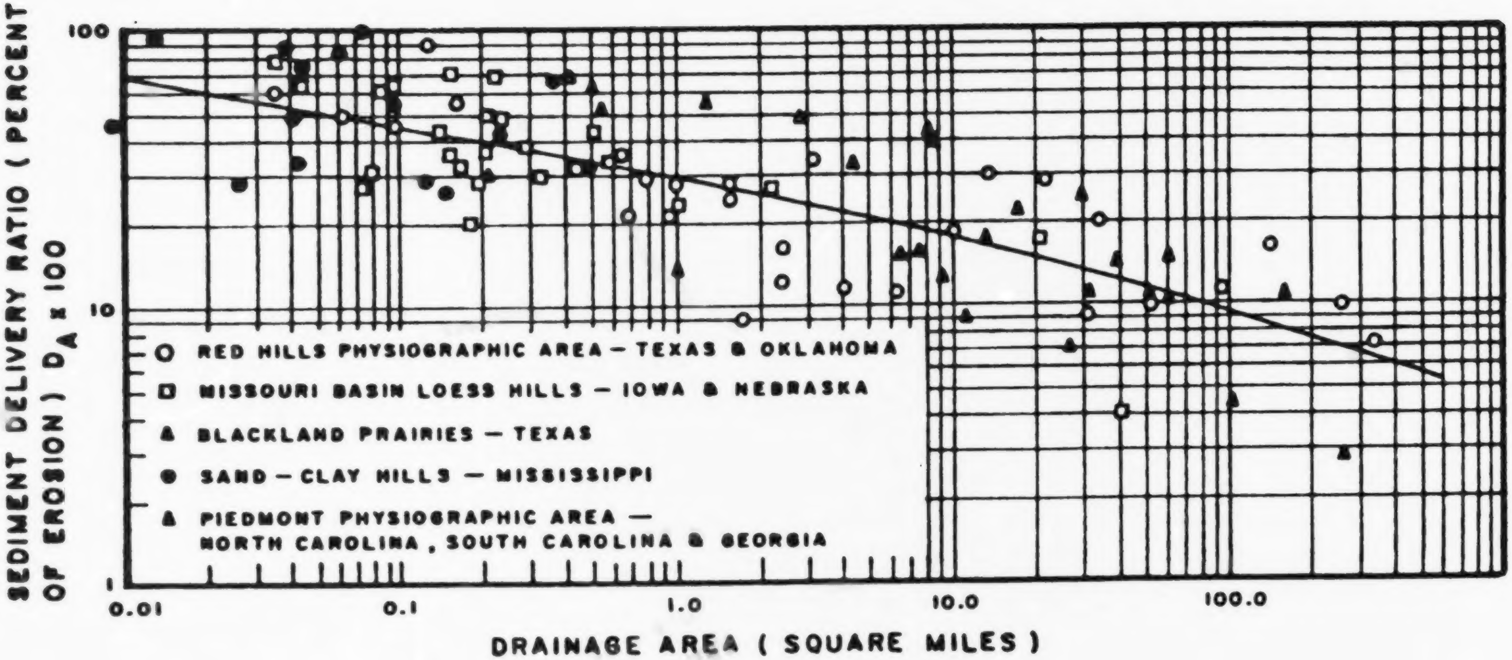


Figure 4. Boyce curve for Da. (Haan and Barfield 1978)

at one. If very dense vegetation is present, the combined factor for D_a and D_v to be used in equation 13 should be between the product of a Figure 4 ordinate and the result of equation 14 and the single value of the lower of the two. Only in the case of very dense vegetation and highly aggregated exposed soils do Haan and Barfield (1978) suggest that D_a times D_v equals the product of a Figure 4 ordinate and the result of equation 14.

A well-channelized watershed will transport relatively large volumes of sediment, while a poorly channelized watershed will provide many opportunities for deposition. This variation in transport capacity is evaluated in equation 13 using D_c, which is determined as a function of a watershed's relief-length ratio. The relief-length ratio is defined as the ratio of elevation difference between the watershed divide at the mainstem and the settling pond site to the length of flow path along the mainstem (Wilson, Barfield, and Moore 1982). Figure 5 was derived by Renfro (1975, cited by Haan and Barfield 1978) and can be used to estimate D_c as a function of relief-length ratio. This curve was prepared for portions of Oklahoma and Texas and therefore must be used with caution in other areas. Haan and Barfield (1978) suggest that if either D_a or D_v is less than one, the value of D_c predicted by Figure 5 should be revised upward to unity.

It is not unusual, particularly for surface coal mines in Alberta, for surface runoff to drain into a working pit from which it is pumped into a settling impoundment. The D_p factor in equation 13 is intended to account for the reduction in delivery ratio resulting from this temporary storage. A crude estimate of this reduction can be made using the following assumptions (Haan and Barfield 1978):

1. All particles settle at their terminal velocity,
2. The water is quiescent, and
3. All particles falling below the pump inlet are trapped.

Figure 6 was based on these assumptions and can be used to estimate the minimum particle size trapped by entering the figure with settling

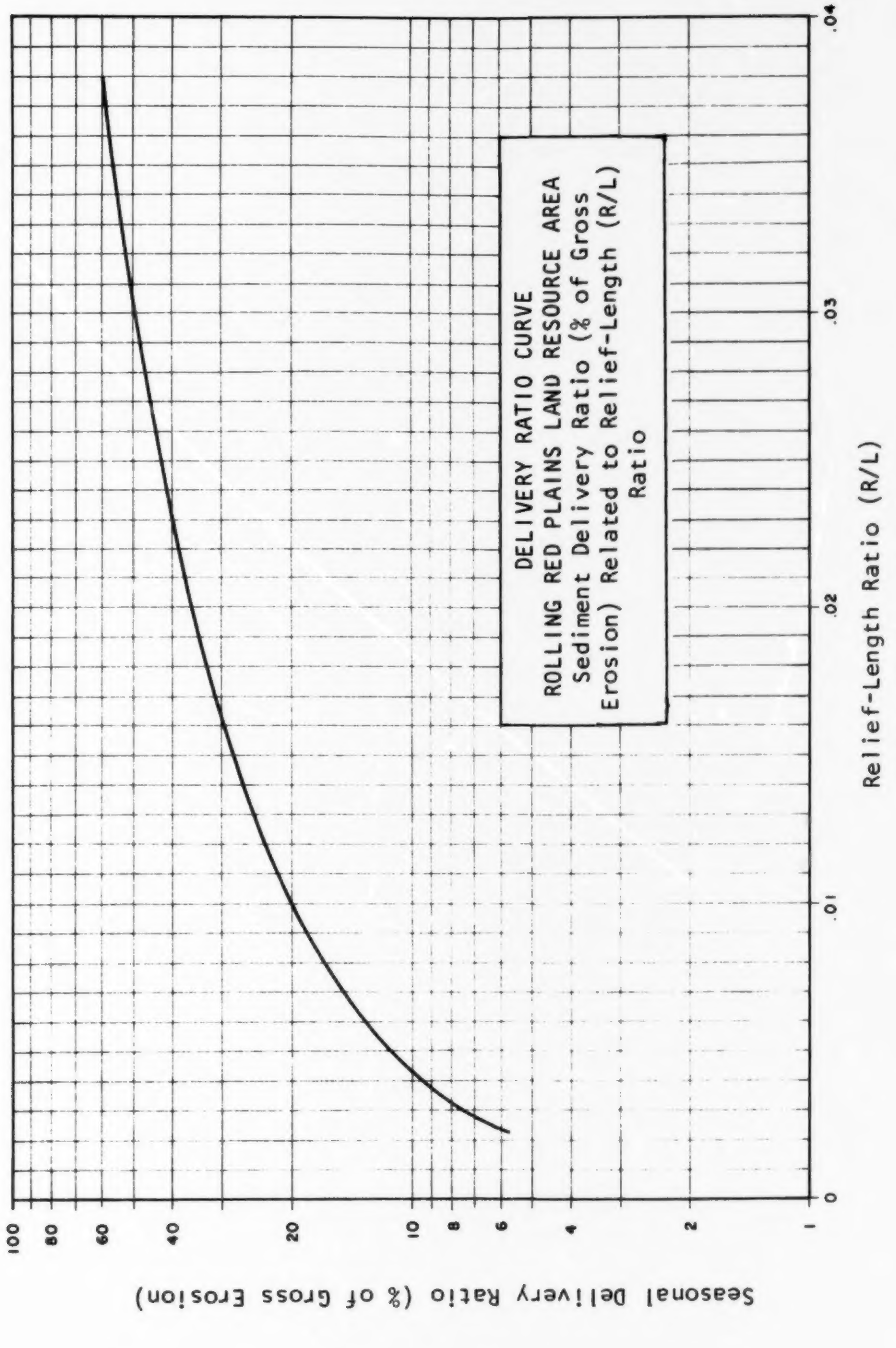


Figure 5. Renfro curve for D_c. (Haan and Barfield 1978)

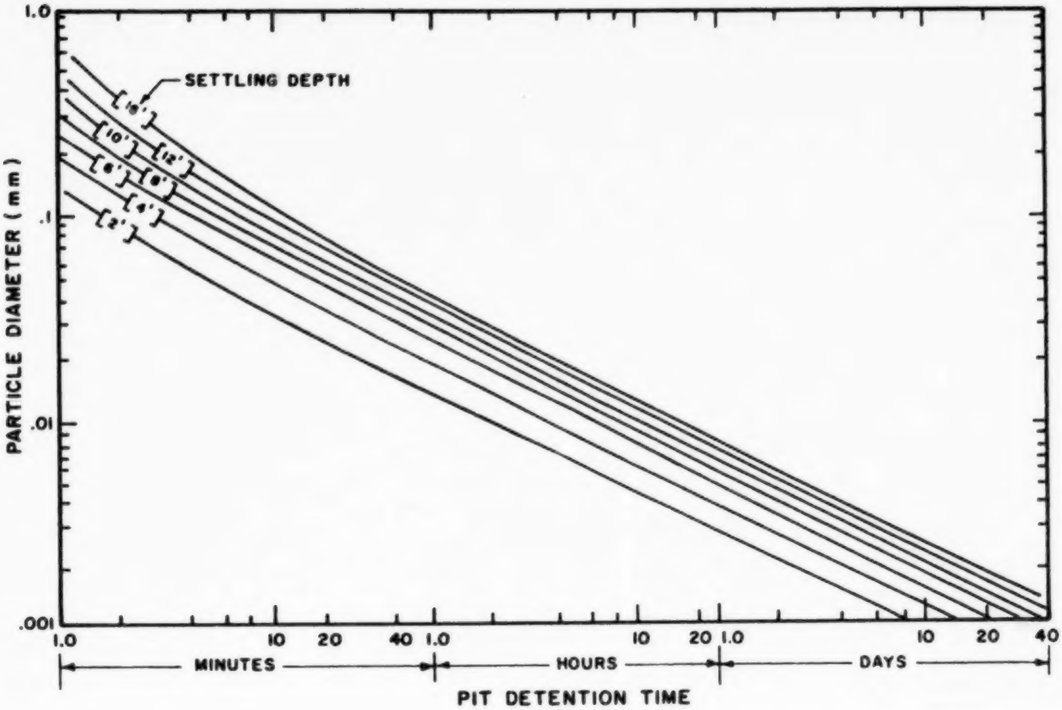


Figure 6. Pit detention time.(Barfield, Warner, and Haan 1981)

depth (depth of water in the pit) and the time of storage since the runoff event (detention time in the pit). The value of minimum particle size trapped is then used along with an estimate of the particle size distribution to predict D_p.

2.5.1.3 Sediment yield model. The USLE - delivery ratio concept described in the preceding section is an empirically based model that is commonly used to estimate average annual sediment production. An alternative to this approach can be provided by the use of one of a number of sediment yield models. Sediment yield models are primarily physically based and are normally used to predict sediment yield from a single storm event. These single storm event yields can then be used to provide relatively accurate estimates of annual average sediment yield.

One of the more well-known single storm event sediment yield models was based on a modification of the USLE (Williams 1975). The modified USLE, or MUSLE, predicts sediment yield by using a runoff erosivity factor instead of the rainfall erosivity factor R (Wilson, Barfield, and Moore 1982). Williams' modified USLE took the following form:

$$Y = 95\ (Q\ q_p)^{0.56}\ K\ LS\ C\ P \qquad (15)$$

where Y is sediment yield in tons, Q is the volume of runoff in acre-feet, q_p is the peak flow rate in cubic feet per second, and the other terms are as defined in the USLE. Equation 15 was based on data derived from 778 individual storm events over watersheds ranging in size from 1.0 ha to 1800 ha with average slopes varying between 0.94% and 5.9%. Care should be exercised when attempting to use the MUSLE on watersheds that differ drastically from those upon which the equation was based. This may limit the applicability of the MUSLE for the Alberta front ranges where slopes may exceed 6%.

A second single storm event sediment yield model has recently been developed that is more adaptable to steep slopes than the MUSLE. The algorithm 'SLOSS' is incorporated in a watershed hydrology model known as SEDIMOT II (see Section 2.7) where it can be used as an alternative to the MUSLE. SLOSS has a more physically realistic basis than the MUSLE and also includes an algorithm that predicts the impact of watershed slopes greater than 20%. Its major drawback at the present time is that it has yet to be calibrated with observed erosion data. Details of the SLOSS subroutine can be found in Wilson, Barfield, and Moore (1982).

In most situations the accuracy required for sediment storage estimates would not justify the time and expense associated with the use of a sediment yield model like the MUSLE or SLOSS. These models are more commonly used in watershed hydrology models (see Section 2.7) to predict the sediment load on a pond during a design rainfall event.

2.5.1.4 Reservoir survey method. One relatively straightforward method of estimating sediment storage requirements involves the observation of sediment yields from watersheds similar to the one for which a particular impoundment is proposed. When a new impoundment site is identified, the measured loss of sediment storage capacity at the closest existing reservoir in a similar watershed is used to derive the sediment loading for the new pond. In the United States this process is expedited considerably by the active reservoir survey program undertaken by the U.S. Soil Conservation Service (SCS). Under this program, thousands of reservoirs are surveyed annually to determine the quantity of sediment deposited and the characteristics of the impoundment watersheds. It is unlikely that similarily comprehensive records could be found for designing ponds in Alberta, particularly those in remote areas, but in the case of large and protracted land disturbing activities (surface coal mining for example) this kind of information could be developed from operating data for the first pond built for a long-term operation. The sediment storage volume for any subsequent ponds could then be sized using this data.

When selecting an existing reservoir to derive a sediment loading rate for a proposed reservoir it is important to be cognizant of any differences in the rainfall records for the two locations. It is also important to ensure that the physical characteristics of the comparison watershed are similar to those of the proposed watershed insofar as they affect sediment yield. The more important characteristics to compare are area, relief ratio, vegetation cover, and portion of area disturbed (Barfield, Warner, and Haan 1981).

2.5.1.5 Density of deposited material. To convert the calculated sediment yield into a volumetric quantity that can be used to size a pond's sediment storage requirements, one must know the density of the deposited material. This density will vary depending on the relative proportions of sand, silt, and clay in the incoming wastewater. Sediment particles are normally classified as one of these three materials as a function of their diameter. The following criteria are typical of those used to characterize sediment particles (Vandivere 1980):

1. Sand - greater than 63 microns,
2. Silt - between 2 and 63 microns, and
3. Clay - less than 2 microns.

Generally speaking, higher proportions of coarse particles in a feedwater will result in relatively high densities for the deposited sediment.

Unfortunately there is limited information in the literature correlating estimated sediment characterizations to in-pond sediment densities. Schroeder and Shields (1983) report that a polymer flocculated influent with high proportions of clay settled to a 'sludge' that exhibited a density of 5% solids by weight at the water-solids interface. It was also reported that this density increased at a rate of about 8% solids by weight per metre of sediment accumulation. Nawrocki and Pietczak (1976) suggest that the in-situ density of deposited sediments is normally in the order of 10% solids by weight. Escher, Kicinski, and Ninesteel (1983) report that neutralized acid mine drainage sludges can be expected to have in-pond densities ranging from 0.5% to 4.5% solids by weight. Unfortunately, neither of these later references provided any correlation to the incoming sediment characterization. Monenco's experience with settling ponds in the Alberta plains region indicates that for incoming sediments with high proportions of clays (i.e., greater than about 50%), impounded sludge densities range between 0.5% and 4% solids by weight depending on the length of time sediments have been retained in a pond.

When designing a new pond, it would be most desirable to establish expected sludge densities by measuring those associated with any other impoundments in the area. When this is not possible, it would be prudent to assume that in-pond densities will not exceed 5% solids by weight unless unusually large portions of the incoming sediment consist of silt and sand.

Once the design sludge density has been established, a settling pond's sediment storage requirements can be determined using the following formula:

$$V_S = 0.1\, Y_L/d \qquad (16)$$

where V_S is the required sediment storage in m^3, Y_L is the sediment yield over the life of the reservoir in Kg, and d is the density of deposited material in % solids by weight. It should be noted that this formula assumes that the specific gravity of sludge is 1.0. The actual sludge density will be greater than 1.0, but for the densities normally encountered in settling pond deposits, the error introduced by this assumption is not significant.

2.5.2 Permanent Pool Volume

A permanent pool is that volume of water above the sediment storage volume and below the crest of the emergency spillway that is not dewatered after a storm. The desired size of the permanent pool cannot be fixed by a theoretical analysis. That is, there is no unique value for the permanent pool volume for a given pond. The volume of permanent pond, the detention volume, and the size of the principal spillway all combine to determine trapping efficiency and effluent sediment concentration from a reservoir. For a given structure, several combinations of permanent pool, detention volume, and principal spillway size will yield the same trapping efficiency or effluent concentration.

Some regulatory authorities require a minimum sized permanent pool as a mechanism for protecting the deposited sediment from resuspension. Algorithms for predicting the effects of permanent pools on effluent sediment concentrations are included in most pond sedimentation models and are discussed more fully in Section 2.6.

2.5.3 Detention Volume

Sediment pond literature in recent years has focused on procedures for sizing the detention volume and determining its effects on effluent concentration and trapping efficiency. Procedures range from simple hydraulic considerations such as detention time designs to complex models that consider diffusion, advection, thermal gradients, and resuspension.

The size of the detention volume is typically determined by using one or more of the criteria listed in Table 7. Design models that can be used to satisfy these criteria are discussed in subsequent sections.

2.5.3.1 Sediment removal efficiency. Sediment removal efficiency, or reservoir trapping efficiency, is the fraction of sediment in a given storm that is trapped. Reservoir design for trapping efficiency requires the modelling of an inflow sediment yield to the reservoir and the use of a trapping efficiency model. Models that can be used for making both of these calculations are discussed in Section 2.6.

A design based solely on sediment removal efficiency will not account for the effects of site-to-site variations in incoming sediment load. For example, a pond with a 95 percent trapping

Table 7. Design criteria used for sizing detention volume in sediment ponds.

Methodology	Remarks
1. Sediment Removal Efficiency	Fraction of sediment trapped during storm. Not a good method for evaluating effluent. If the inflow concentration is 100 000 mg/L, a one percent change in trapping efficiency gives a 1000 mg/L change in effluent concentration.
2. Detention Time	Reservoir sized to yield average detention time for the entire storm. Early inflows will have a much smaller detention time than the average value and will thus have effluent sediment concentrations much higher than would be predicted based on the average detention time. Permanent pool volume must be accounted for.
3. Total Suspended Solids	Reservoir sized for average or peak suspended solids concentration. Includes both settleable solids and colloidal particles.
4. Settleable Solids	Reservoir sized for an average or peak settleable solids concentration. Settleable solids do not include the colloidal size particles.
5. Total Sediment	Reservoir sized for a maximum sediment load that can be discharged in a design storm or over a specified period of time.

efficiency could have the following average outflow concentrations for the given inflow concentrations:

Reservoir	Trapping Efficiency (%)	Average Inflow Concentration (mg/L)	Average Outflow Concentration (mg/L)
1	95	10 000	500
2	95	100 000	5 000
3	95	200 000	10 000

Although all three reservoirs have identical trapping efficiencies, the effluent concentrations are considerably different. It can be concluded therefore, that trapping efficiency in itself is not a good design criteria if the objective of the design is to satisfy an effluent suspended solids limitation.

2.5.3.2 Detention time. Settling ponds that are designed simply to provide a particular detention time are sized on the basis of the hydraulic properties of the pond and its inflow. The design procedure is therefore an iterative one in which a pond size and configuration are selected and then considered in relation to the design flow or storm event to predict its detention time. The pond configuration is modified until it provides the minimum required detention time.

The detention time in an impoundment and the methods used to estimate it vary not only with the characteristics of the contributing watershed but with the type of land disturbing activity encountered. For many activities, substantial volumes of runoff accumulate in a working area or pit that is not self-draining. This water, along with any groundwater that may accumulate in the same area, is normally pumped into the impoundment, resulting in a pond inflow that is controlled to some extent. On the other hand, a sediment pond may be located below the land disturbing activity on one of the watershed's natural drainage courses. In this case, the pond inflow is essentially uncontrolled and can be modelled by the hydrograph applicable for the storm event and watershed location in question. The following discussion outlines the various methodologies that can be used to estimate detention time for either controlled (pumped) or uncontrolled (storm) flow.

2.5.3.2.1 Uncontrolled (storm) flow. For a pond with ideal steady state flow, the detention time is given by

$$Td = \frac{V}{Q} \tag{17}$$

where V is the volume of the pool and Q is the steady state flow rate. For storm flows, which are not steady state, the definition is not so simple. The plug flow concept is probably the most logical procedure to be used in this case, but also the most complicated numerically. To use the plug flow concept, the inflow hydrograph is divided into plugs of equal volume as shown in Figure 7. These plugs are routed through the reservoir and the detention time for each plug determined. The theoretical detention time for the storm is the average detention time for the plugs, or

$$T_d = \frac{1}{n} \sum_{i=1}^{n} Td_i \tag{18}$$

where Td_i is the detention time of plug i and n is the number of plugs.

Determination of theoretical detention time by the plug flow concept requires considerable computation time. An alternate definition of detention time uses the centroid concept. Theoretical detention time using the centroid concept is defined as the time difference between the centroid of the inflow and outflow hydrographs, or

$$T_d = T_{mo} - T_{mi} \tag{19}$$

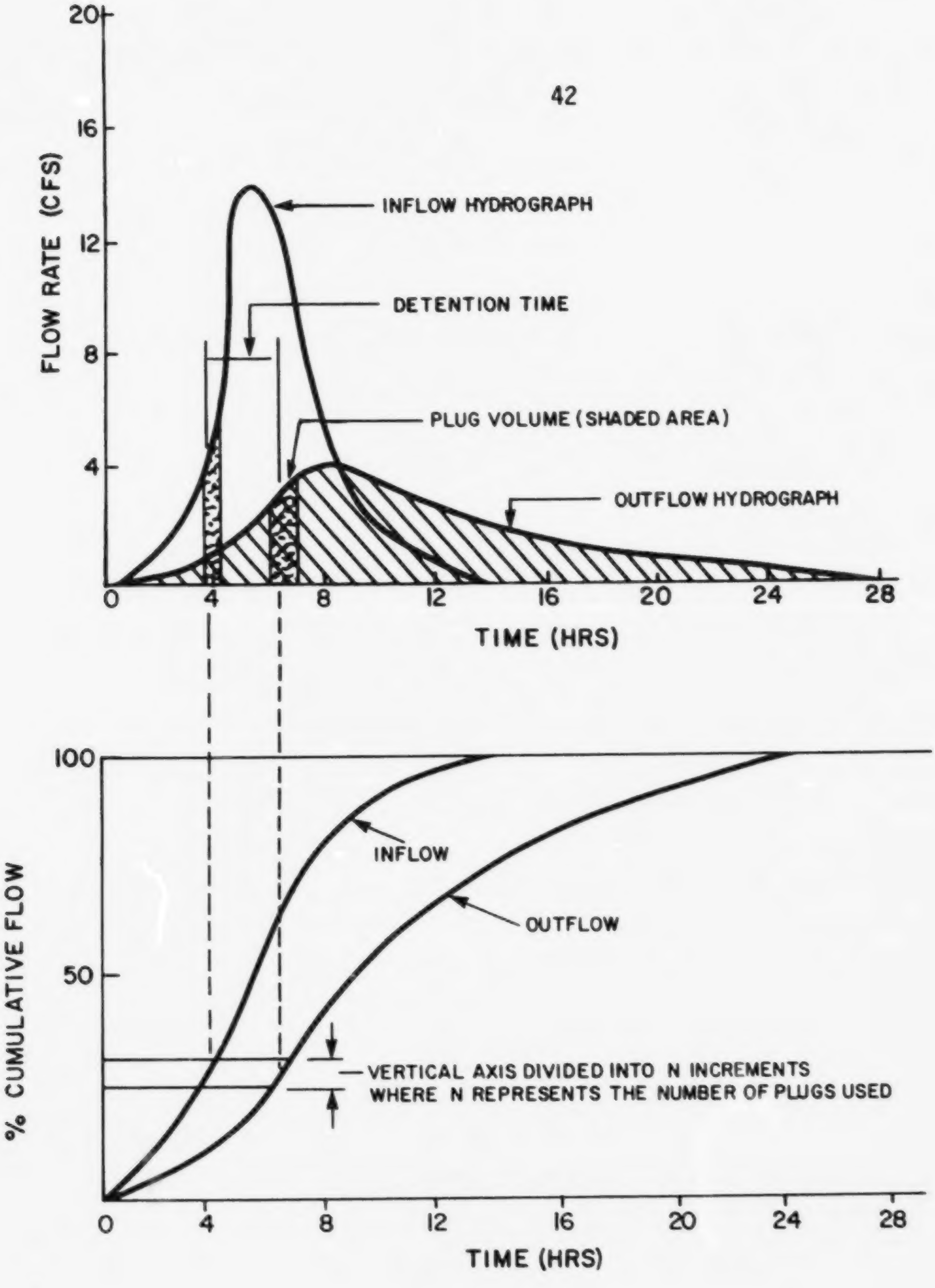


Figure 7. Illustration of the method for determining the detention time for plug of flow.

where T_{mo} and T_{mi} are the time to the centroids of the outflow and inflow hydrographs, respectively. This can be determined by dividing the inflow and outflow hydrographs into n increments of time lengths t_i and t_o as shown in Figure 8. The time to the centre of mass of the inflow hydrograph can be calculated from

$$T_{mi} = \frac{\sum_{j=1}^{n} V_{i,j} T_{i,j}}{\sum_{j=1}^{n} V_{i,j}} \tag{20}$$

or

$$T_{mi} = \sum_{j=1}^{n} \frac{(Q_{i,j} + Q_{i,j-1})\ (t_{i,j} + t_{i,j-1})\ \Delta t_{\ i,j}}{4V} \tag{21}$$

The time to the centre of mass of the outflow hydrograph can be calculated from

$$T_{mo} = \frac{\sum_{j=1}^{m} V_{o,j}\ T_{o,j}}{\sum_{j=1}^{m} V_{o,j}} \tag{22}$$

or

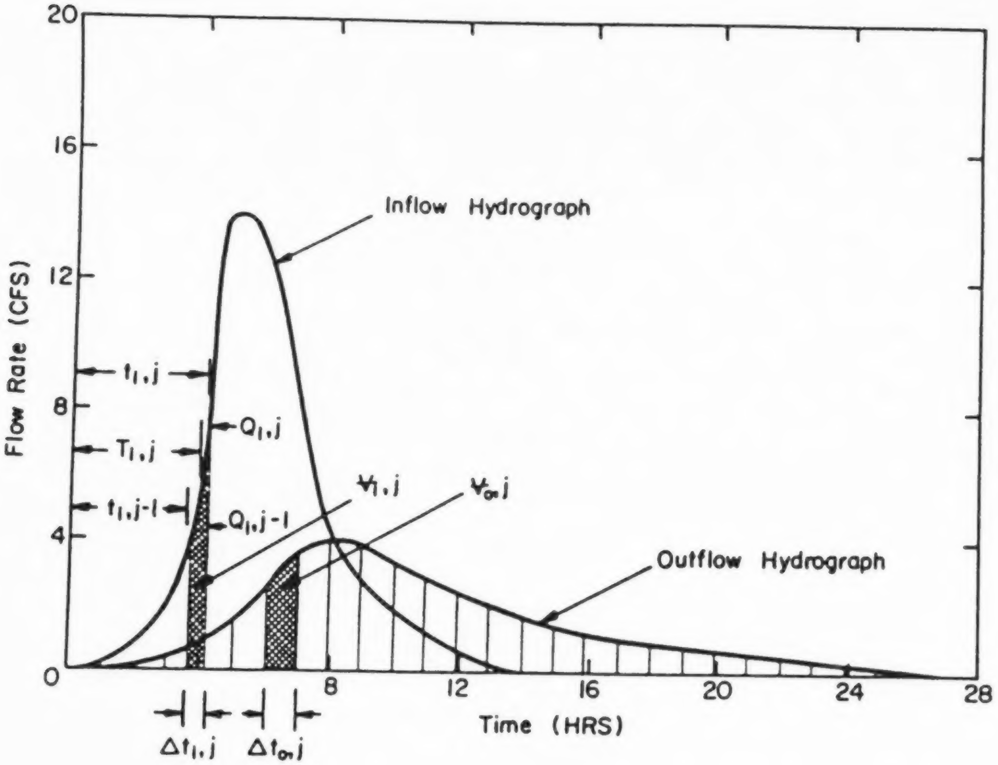


Figure 8. Illustration of the variables used in calculating detention storage time. The variables $T_{0,j}$, $t_{o,j}$, and $t_{o,j-1}$ are not illustrated, but relate to the incremental outflow volume $\forall_{o,j}$ in the same manner as $T_{i,j}$, etc. relate to $\forall_{i,j}$.

$$T_{mo} = \sum_{j=1}^{m} \frac{(Q_{o,j} + Q_{o,j-1})\ (t_{o,j} + t_{o,j-1})\ \ t_{o,j}}{4V} \tag{23}$$

Note that in equations 20 to 23, the number of increments in the inflow hydrograph n, does not necessarily have to be equal to that in the outflow hydrograph m. If the time increments t_i are equal to t_o, then they cannot be equal. To accurately estimate the time between the centroids of inflow and outflow hydrographs using the above procedure, it is necessary to (Ward, Barfield, and Tapp 1979):

1. Develop an inflow hydrograph,
2. Select a spillway design for a particular reservoir location, and
3. Route the flow through the structure to obtain an outflow hydrograph.

Equations 20 through 23 are then used to calculate the time to the centroids of the inflow and outflow hydrographs and equation 19 is used to calculate the detention time. If it is found that the spillway size initially selected does not give the required detention time, a new system is designed and the calculations repeated.

Designing a reservoir to have a particular detention storage time can be quite tedious if it is necessary to make all the calculations indicated by equations 20 to 23 for each trial pipe size and storage volume. Barfield, Warner, and Haan (1981) showed that a first estimate can be represented by triangles. The equations given in Figure 9 will then apply (all runoff is in watershed mm). In order to use these equations it is still necessary to develop an inflow hydrograph so that the time to peak t_p, the peak inflow rate q_p, and the runoff volume V can be determined. Through the use of the equations in Figure 9 it is possible to get a first estimate of the storage and peak outflow rates necessary to provide the desired detention time. The design procedure for using the triangular hydrograph method is:

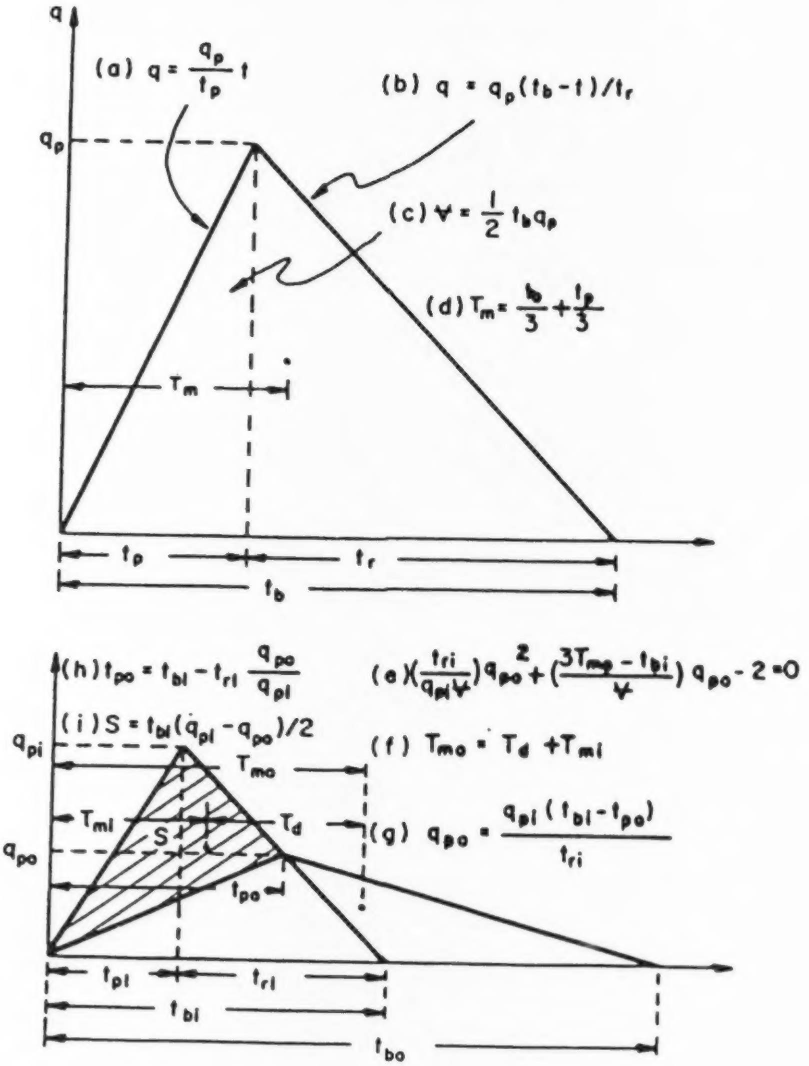

q
(a) $q = \frac{q_p}{t_p} t$
(b) $q = q_p(t_b - t)/t_r$
q_p
(c) $\forall = \frac{1}{2} t_b q_p$
(d) $T_m = \frac{t_b}{3} + \frac{t_p}{3}$
T_m
t_p
t_r
t_b
(h) $t_{po} = t_{bi} - t_{ri} \frac{q_{po}}{q_{pi}}$
(e) $(\frac{t_{ri}}{q_{pi} \forall}) q_{po}^2 + (\frac{3T_{mo} - t_{bi}}{\forall}) q_{po} - 2 = 0$
(i) $S = t_{bi}(q_{pi} - q_{po})/2$
(f) $T_{mo} = T_d + T_{mi}$
(g) $q_{po} = \frac{q_{pi}(t_{bi} - t_{po})}{t_{ri}}$
q_{pi}
q_{po}
T_{mo}
T_{mi}
T_d
S
t_{po}
t_{pi}
t_{ri}
t_{bi}
t_{bo}

Figure 9. Triangular hydrograph assumptions and detention storage time equations. (Barfield, Warner, and Haan 1981)

1. Estimate the time to peak of the inflow hydrograph based on a linear rising limb;
2. From equation (c) (Figure 9) determine the time base and thus the recession time of the inflow hydrograph;
3. From equation (d), determine the time to the centre of mass of the inflow hydrograph;
4. From equation (f), determine the required time to the centre of mass of the outflow hydrograph based on the design detention time;
5. Solve the quadratic equation given by equation (e) for peak outflow;
6. Solve equation (i) for the required storage volume;
7. Based on the stage storage curve, select the maximum stage for the reservoir as shown in Figure 10;
8. Based on the maximum stage and peak discharge, select the maximum pipe size as shown in Figure 10.

Since the triangular hydrograph approach is only an approximation, it is desirable to check the results by detailed routing.

If a settling pond stores a portion of the runoff between storm events (i.e., has a permanent pool), the detention time computations discussed above are no longer valid, and must be altered to account for the effects of the permanent pool. When using the plug flow concept, the effects of stored runoff can be included by assuming that the first water discharged is that of the permanent pool (Ward, Barfield, and Tapp 1977). The detention time of the first plug of flow will then be the time required to displace the permanent pool waters illustrated in Figure 11. The displaced permanent pool volume is equal to permanent volume minus dead storage volume. Dead storage is discussed in Section 2.6.

If the centroid concept is used for design, the total storm detention time becomes

$$T_d = \left[(V_p - V_{ds})T_{dp} + (V - V_p + V_{ds})T_{mo} - T_{mi})\right]/V \qquad (24)$$

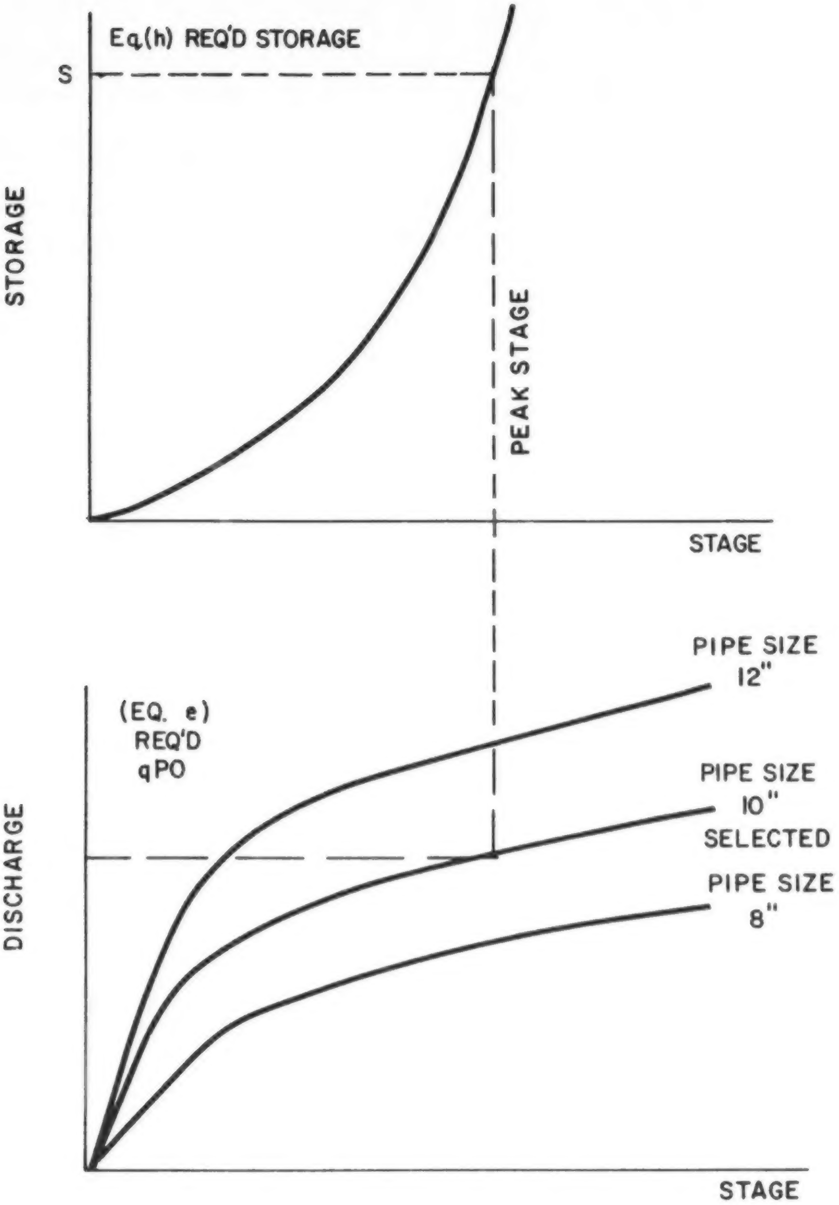


Figure 10. Illustration of reservoir design for detention time.

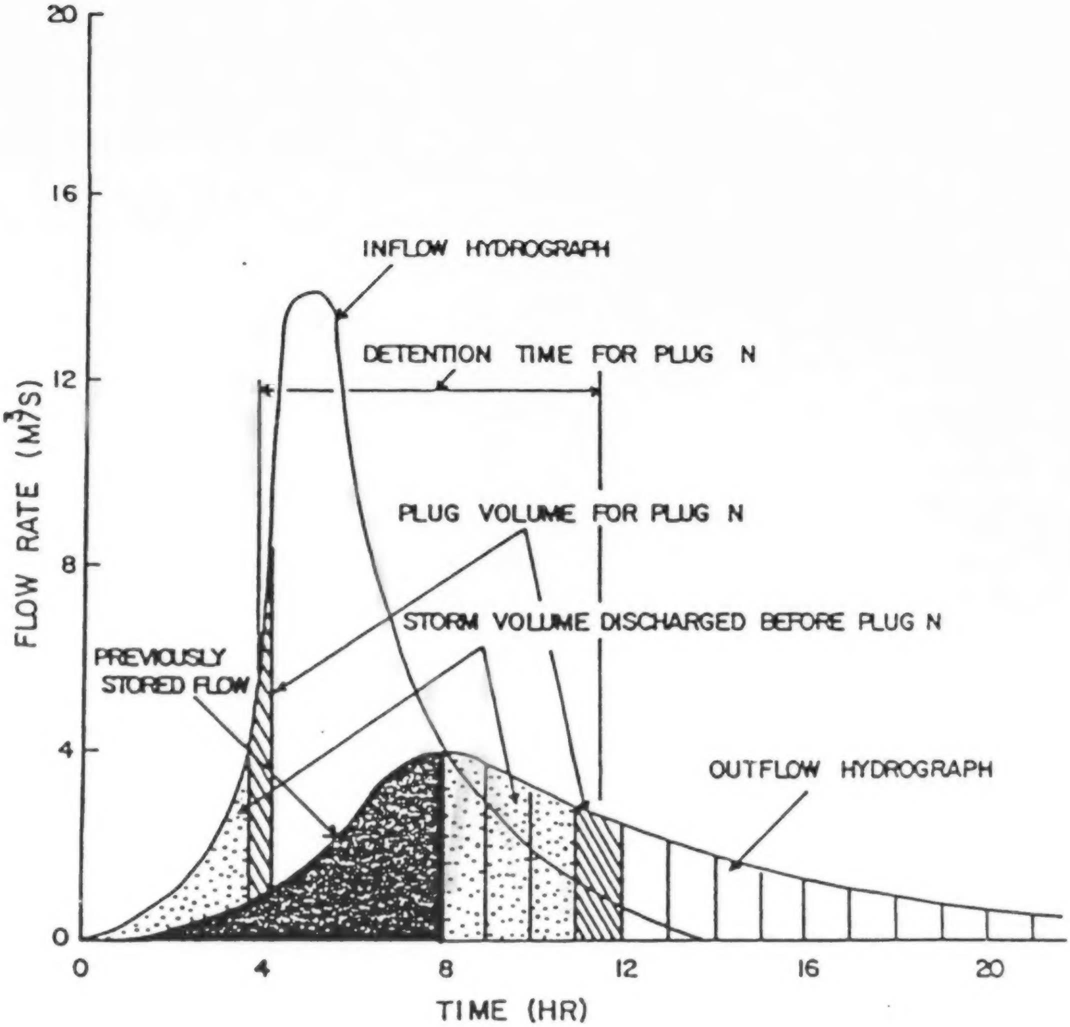


Figure 11. Illustration of the effects of permanent pool on plug flow detention time. (Wilson, Barfield, and Moore 1982)

where V_p is the volume in the permanent pool, V_{ds} is dead storage, T_{dp} is the detention time assigned to the permanent pool, V is the storm runoff volume, and $T_{mo}-T_{mi}$ is the time between the centres of mass of the inflow and outflow hydrographs. The problem with making the computation for permanent pools is that of assigning an acceptable detention time to the water displaced from the permanent pool. Studies using a computer model of reservoir sedimentology known as DEPOSITS (Ward, Haan, and Barfield 1977) indicate that a conservative value is 2.5 to 2.7 times $T_{mo}-T_{mi}$. This value is actually the detention time of the last plug of outflow in a storm.

If a series of settling reservoirs in tandem are being used, no acceptable definition of detention storage time has been established. In this case, it is necessary to select one large reservoir that gives an equivalent trap efficiency to the series of ponds under consideration, with a given detention storage time.

Designs based on detention time consider only the hydraulic properties of a pond and its inflow and not the sediment characteristics of the inflow. Since the trapping efficiency of a given pond depends heavily on settling characteristics, such a design is usually inadequate unless a very conservative detention time is used. An example of the variation in effluent concentration from reservoirs with identical detention times but varying inflow particle size distributions is shown in Table 8. All parameters other than particle size distribution are held constant. The table shows that detention time alone is a poor predictor of final effluent suspended solids concentration.

2.5.3.2.2 Controlled flow. As noted previously, there are many situations in which flow into a settling pond is pumped from a working area that is not self-draining. In this case the pond inflow is said to be controlled and the inlet hydrograph takes a form that is radically different from those described previously.

There are two basic contributors to the flow into a working area or pit: groundwater or base flow, and storm flow. In some

Table 8. Effects of varying particle size on effluent concentration with a constant detention time[a].

Test No	Particle Diameter (mm)			Peak Effluent TSS	Peak Effluent Settleable Solids	
	D_{10}	D_{50}	D_{90}	mg/L	ml/L	mg/L
1	.10	1.00	40.00	3,757	.10	(146)
2	.01	.10	.90	15,394	.83	(1160)
3	.004	.01	.09	85,231	2.63	(3676)

[a] All computations made with SEDIMOT II (see Section 2.7)

Notes: Watershed Characteristics:
Area - 8 ha, Time of Conc. = .50 hours.
Conc. No = 80, Erodibility = .40, Slope = 10%, Slope length = 46 m
Cover factor = .70

Reservoir Characteristics:
Permanent Pool = 4600 m^3
Detention Volume = 1600 m^3
Drop Inlet Spillway = 600 mm diameter cmp.

Predicted Values:
Peak inflow rate = 1.1 m^3/s
Peak discharge = 0.6 m^3/s
Peak inflow sediment concentration - 213,762 mg/L
Average Detention Time (including stored flow) = 11 hours

locations, groundwater can contribute a substantial portion of the total annual volume of flow discharged to an impoundment. Groundwater flow is relatively uniform and its impact on pond effluent quality can be significant, particularly if a 'dry weather' suspended solids limitation has been established by the regulatory authority. The Environmental Protection Agency (EPA) and the Office of Surface Mining (OSM) in the United States are now promulgating such 'dry weather' quality requirements in addition to separate requirements for wet weather conditions (U.S. Department of the Interior 1983).

Calculating pond detention time assuming groundwater is the only contributing flow is relatively straightforward. Because groundwater flow is reasonably constant it can be idealized as steady state flow and detention time can be determined using equation 17. To evaluate a pond for the worst case 'dry weather' situation, the value of Q in equation 17 should be taken as the maximum average pumped flow resulting from in-pit groundwater accumulations anticipated during the life of the impoundment.

For storm flows, the determination of pond detention time under controlled inflow conditions is not so easy. In this case the shape of the reservoir inflow hydrograph is as shown in Figure 12, in which the peak discharge is equal to the pump capacity. The development of a pond inflow hydrograph for controlled flow requires the consideration not only of the design storm and watershed characteristics but also the operational requirements and capabilities of the land disturbing activity. The pit dewatering capability of a particular operation has a profound effect on settling pond inflow and is sized as a function of the maximum time that a pit can be allowed to be inundated after a storm event. If it is economically desirable to dewater a pit as quickly as possible, the operator will desire to have, or have ready access to, a large pit pumping capacity. In order to develop a pond inflow hydrograph for controlled flow conditions after a storm event, the following information is required:

1. The inflow hydrograph for the pit sump. This can be developed using the conventional procedures described

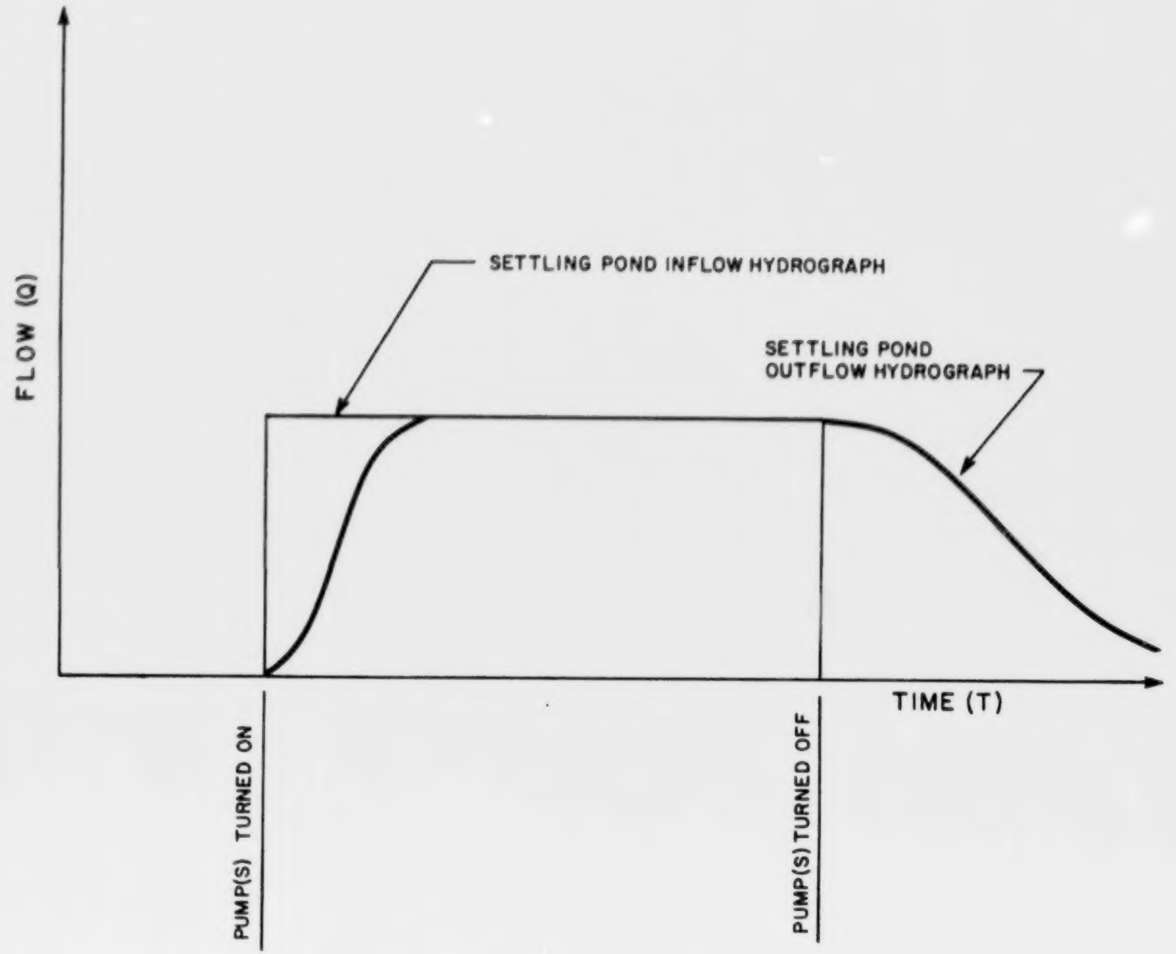

FLOW (Q)
SETTLING POND INFLOW HYDROGRAPH
SETTLING POND
OUTFLOW HYDROGRAPH
TIME (T)
PUMP(S) TURNED ON
PUMP(S) TURNED OFF

Figure 12. Controlled flow hydrographs.

in Section 2.3 or by using a computer watershed model such as SEDIMOT II (see Section 2.7);

2. The number and capacity of pumps that an operator has at his disposal; and
3. The most likely scenario for pump operation during a design storm event. This will be a function of the maximum period of time that the operator feels a land disturbing activity can be interrupted due to pit inundation. It is unlikely that an operator would have at his immediate disposal all the pumps that would be necessary to dewater the pit as quickly as would be desired; therefore, some time would be required to procure all of the additional pumps needed. The resulting time lag would affect the shape of a pond's inflow hydrograph and consequently should be estimated and accounted for.

The basic procedure for developing a pond inflow hydrograph from the above information is to:

1. Route the pit sump inflow hydrograph into the working area to determine the stage versus time relationship in the pit;
2. Establish point 'A' on Figure 13 using the pit stage versus time relationship. Point 'A' is defined as the time at which the water stage in the pit reaches a level that interrupts pit operation. The location of point 'A' is highly site specific and will depend heavily on the pit operator's assessment of the maximum allowable water level for a working pit. It should be noted that point 'A' need only be defined if it is necessary to know the total time that elapses between initiation of a storm event and the resumption of pit operations. The shape of the pond inflow hydrograph is a function of the volume of runoff and the operating scenario for the pumps and is unaffected

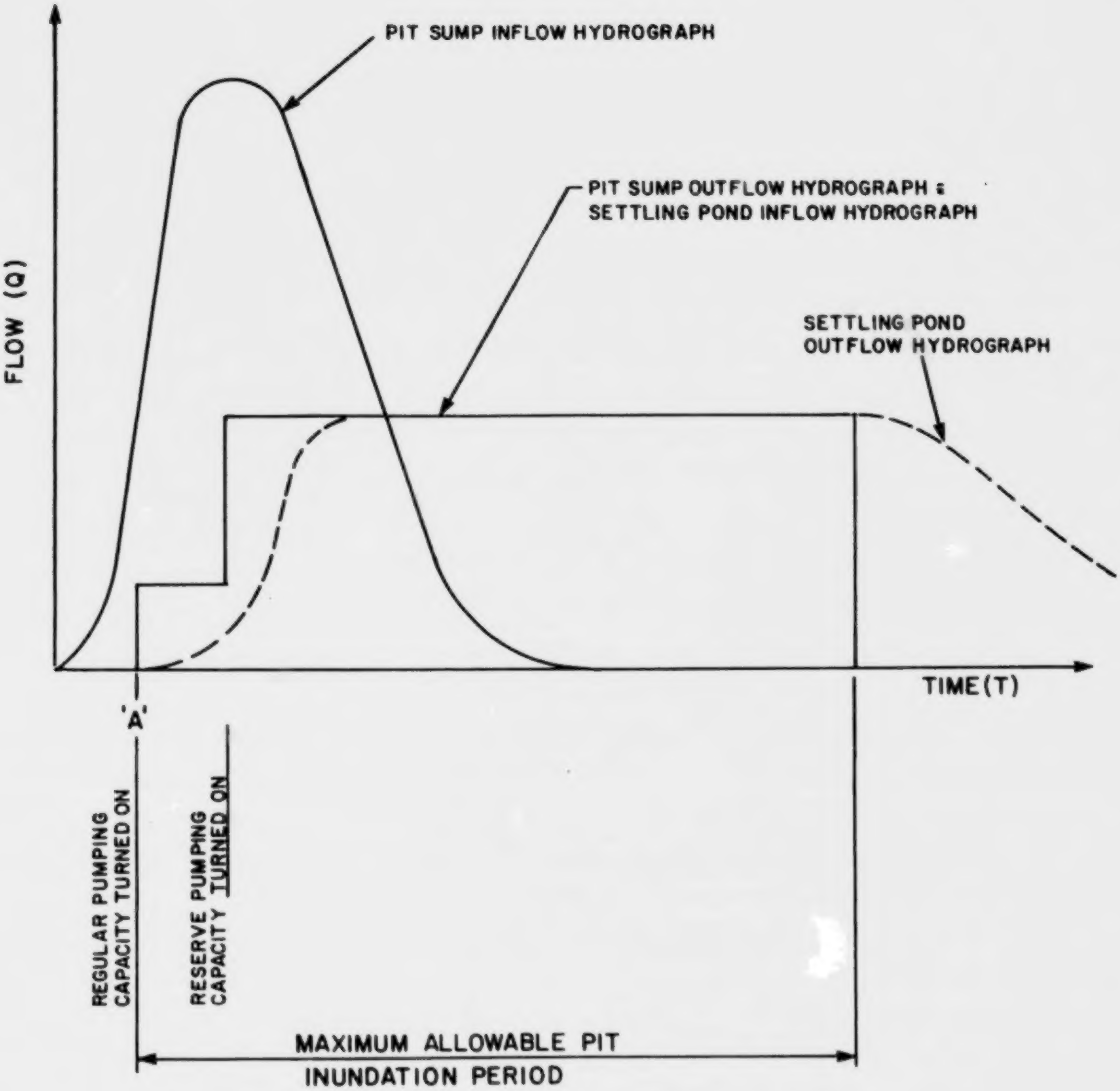

PIT SUMP INFLOW HYDROGRAPH
PIT SUMP OUTFLOW HYDROGRAPH ≡ SETTLING POND INFLOW HYDROGRAPH
SETTLING POND OUTFLOW HYDROGRAPH
FLOW (Q)
TIME(T)
'A'
REGULAR PUMPING CAPACITY TURNED ON
RESERVE PUMPING CAPACITY TURNED ON
MAXIMUM ALLOWABLE PIT INUNDATION PERIOD

Figure 13. Settling pond hydrographs for controlled flow conditions.

by the time the pumps are initially turned on. If it is desired simply to estimate settling pond performance during a storm event it is not necessary to determine the pit stage versus time relationship or to establish point 'A'.

3. Plot the maximum allowable pit inundation period on the abscissa of the hydrograph.
4. Establish a pump operating scenario such that the area of the pit outflow hydrograph (assumed to be equal to the settling pond inflow hydrograph) during the maximum allowable inundation period is equal to the area of the pit inflow hydrograph. This would be done by considering the number and capacity of pumps available and the time at which each pump is turned on.

The above procedure neglects the effect of any base (groundwater) flow that may occur prior to and during a storm event. It is assumed that these base flows would be negligible relative to those encountered during a storm event and consequently that the error introduced by neglecting them would be insignificant. The procedure also assumes that pump discharges are delivered to the settling pond via a hose or pipeline. If the pumps actually discharge to a ditch upstream of the pond, a more accurate analysis would require that this flow be routed through the ditch using a procedure such as Muskingham's method (see Linsley, Kohler, and Paulhus 1975).

Once the settling pond inflow hydrograph has been prepared, the flow can be routed through the reservoir to develop the pond outflow hydrograph. The detention time can then be determined using the plug flow concept or by calculating the time difference between the centroids of the inflow and outflow hydrographs. For obvious reasons, the simplified triangular hydrograph approach discussed earlier cannot be applied to the inflow hydrograph shapes associated with controlled flow conditions.

It should be noted that if the total runoff volume from the design storm event is much greater than the total settling pond volume (excluding sediment and dead storage), a reasonable approximation of detention time can be made by assuming steady state flow and using equation 17 (i.e., Td = V/Q). Q in this case would be equal to the maximum flow with all pumps operating.

2.5.3.3 Total suspended solids (TSS). In the past one of the most common design approaches for sizing the required detention storage has been to provide that volume that satisfies an effluent suspended solids limitation. Total suspended solids concentration (TSS) is the mass of total sediment in suspension per unit volume of water and is typically measured in units of mg/L. This design approach owes its popularity to the fact that most authorities regulate discharges from land disturbing activities by placing specific suspended solids limitations on discharged effluent. The regulations promulgated until recently by the Office of Surface Mining (OSM) in the United States were typical of these kinds of regulations. The OSM guidelines stated that pond effluent suspended solids concentrations should not exceed 35 mg/L average and 70 mg/L peak during a 10-year, 24-hour storm. The existing requirements for the coal mining industry in Alberta (see Table 1) are similar in character to the OSM guidelines except that consideration is given to the natural background TSS concentrations for watercourses in the immediate vicinity of the mine. The Alberta guidelines allow effluent TSS concentrations to rise above the 50 mg/L level if the natural background concentration for storms less than the 10-year, 24-hour storm event exceeds 40 mg/L.

A pond can be designed to meet a specific average effluent suspended solids concentration using the equation

$$C = \frac{Y(1-E)}{Q} \times 10^3 \qquad (25)$$

where C is the average effluent concentration in mg/L, Y is the sediment yield in kg, Q is the runoff volume discharged from the pond in m^3, and E is the fraction of sediment trapped. The various models that can be used to predict E are described in detail in Section 2.6. Designing a pond to meet a peak outflow TSS limitation is considerably more complicated and usually requires the use of a computer. Procedures for completing the necessary calculations are also described in Section 2.6.

Recent studies in the United States have shown that there are difficulties associated with design approaches for detention volume that are based on specific effluent TSS limitations. In a paper study using the University of Kentucky DEPOSITS model (see Section 2.6.2.3) (Ettinger and Lichty 1979), the U.S. Environmental Protection Agency (EPA) showed theoretically that the American OSM 35 and 70 mg/L criteria could not be met with any reasonable size basin. At that time it was proposed that the use of flocculation would make it possible to meet the 35 and 70 mg/L effluent criteria. Studies by Tapp, Barfield, and Griffin (1981) in a pilot size pond showed that the use of flocculants at remote mine sites with no power or attendant available would not always bring the effluent quality into compliance. In a study of six sediment ponds in Pennsylvania, Maryland, and West Virginia, Poe, Betson, and Singh (1983) found that well-designed ponds did not meet the OSM 35 and 70 mg/L suspended solids standard even for flows less than the 10 year storm event. In an earlier study, Kathuria, Nawrocki, and Becker (1976) also found that sediment ponds designed according to required standards did not meet the 35 and 70 mg/L limitation.

2.5.3.4 Effluent settleable solids. As a result of the studies noted above, the USEPA and OSM began promulgating regulations that require detention volume determinations to be based on effluent settleable, rather than suspended, solids limitations. The settleable solids concentration is defined as that volume of particles that settles to the bottom of an Imhoff cone during one hour of quiescent settling. Since the volume of an Imhoff cone is one liter, settleable solids figures are typically reported in mL/L. A settleable solids test provides a measure of the concentration of larger (greater than about 10 microns) particles in an effluent and does not include colloidal solids which are so small that they remain in suspension due to Brownian movement. The revised standards that were adopted by the EPA and OSM are summarized in Table 9.

The procedure for sizing detention volume based on settleable solids is essentially the same as that previously described for suspended solids. The sedimentation models that can be used to predict effluent settleable solids are described in detail in Section 2.6. In using these models it is often desirable to convert effluent TSS to settleable solids. Barfield, Warner, and Haan (1981) proposed an equation to make this conversion. In deriving this equation it was assumed that the volume of settleable solids is small relative to the total volume and that the particles are uniformly distributed at the start of the test. The resulting equation is

$$SS = \frac{C}{W}\left[(1-X_o) + \sum_{i=1}^{X_o} k\, V_s i^3\, \Delta X_i\right] \tag{26}$$

where SS is settleable solids in mL/L, C is the concentration of total suspended solids in mg/L, W is the dry bulk density of solids in mg/mL, X_o is the fraction of particles finer than a particle that will settle from the top of the Imhoff cone shown in Figure 14 to the bottom in one hour (0.011 mm at 20°C), V_{si} is the settling velocity of a particle of size class i, X_i is the fraction of particles of size class i, and k is a constant equal to

$$2.135 \times 10^{10} \quad \text{sec}^3/\text{ft}^3 \tag{27}$$

This procedure is only valid for low effluent TSS concentrations. Wilson, Barfield, and Moore (1982) discuss a more complicated procedure that is valid at high effluent concentrations.

In order to make a settleable solids calculation, it is necessary to have estimates of the effluent particle size

Table 9. Revised USEPA and OSM quality requirements for mine drainage.

Pollutant or pollutant property		Maximum for any one day[d]	Average of daily values for 30 consecutive days[d]
1. Dry weather conditions[a]			
Total Iron	(mg/L)	7.0	3.5
TSS	(mg/L)	70.0	35.0
pH		(c)	(c)
2. Wet weather conditions[b]			
Settleable solids	(ml/L)	0.5	-
pH		(c)	(c)

[a] Dry weather conditions apply to discharges which are not the result of a precipitation event

[b] Wet weather conditions apply to any discharge or increase in the volume of a discharge caused by precipitation within any 24-hour period less than or equal to the 10-year, 24-hour precipitation event (or snowmelt of equivalent volume).

[c] Within range of 6.0 and 9.0 at all times.

[d] In case of a precipitation event exceeding the 10-year, 24-hour storm, only the requirements for pH apply.

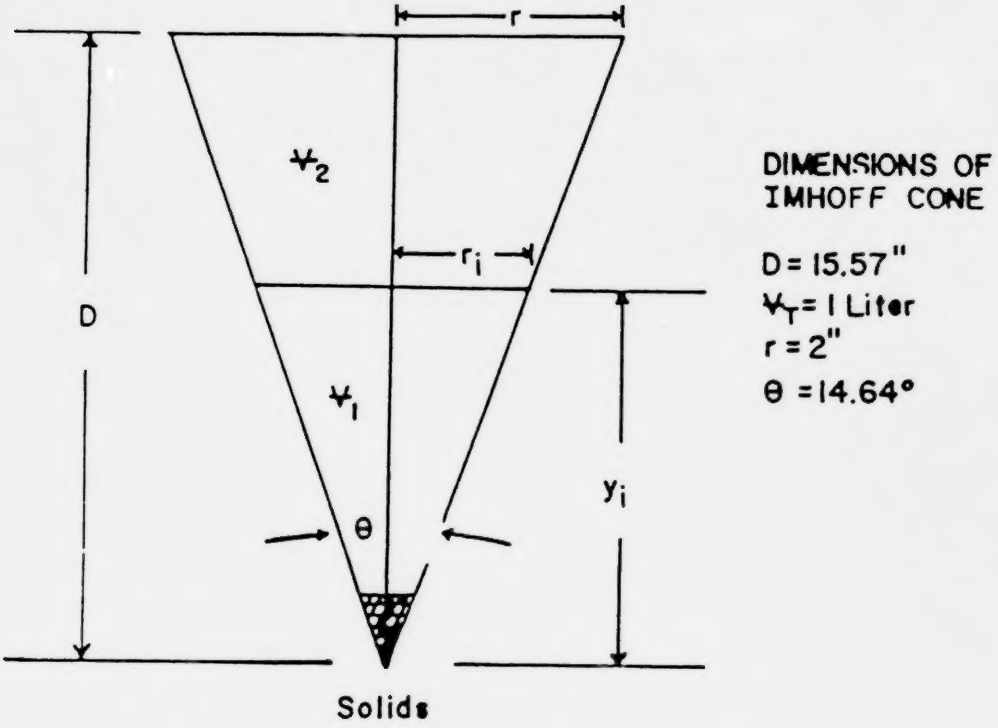

r
$\forall_2$
r_i
D
$\forall_1$
θ
y_i
Solids
DIMENSIONS OF IMHOFF CONE
D = 15.57"
$\forall_T$ = 1 Liter
r = 2"
θ = 14.64°

Figure 14. Derivation sketch for settleable solids equations using an Imhoff cone. (Barfield, Warner, and Haan 1981)

distribution, the effluent TSS, and the bulk density of the settled material. Effluent TSS and size distribution can be calculated from one of the hydrology and sedimentology models discussed in Section 2.6. The difficulty in estimating bulk density represents the major drawback of equation 26. Compression settling results in a variation of bulk density with time and therefore it is necessary to select a W value for equation 26 that is representative of the bulk density of sediment in the bottom of an Imhoff cone after one hour.

The settleable solids criteria may be a workable solution for many situations where downstream conditions are not sensitive to colloidal sediments. It provides for collection of the material that can be trapped by normal gravitational settling and allows for the design of reasonably sized ponds. However, in locations where downstream areas support significant populations of aquatic life, the high suspended solids concentrations that can be present at a settleable solids level of 0.5 ml/mL (see Table 7) may require the maintenance of a TSS standard.

2.5.3.5 Total design storm.

A simple design procedure that is used in some areas where a suspended solids criteria must be met is to store the design storm and release it over a period of days after runoff has stopped. This technique is used more frequently under arid conditions where no base flow is likely and where runoff events occur infrequently.

2.5.4 Flood Storage Volume

The return period of a design storm for sediment control will typically be between 2 and 10 years. A reservoir that only has sufficient volume to safely pass this storm will overtop if an event with a much longer return period occurs during the life of the system. Such over-topping can result in a washout of pond embankments with subsequent downstream flooding. In order to prevent overtopping, an emergency spillway is designed to pass the rare event through a channel constructed in stabilized soil. This stabilized channel should not be constructed through the reservoir fill.

The return period required for the emergency spillway design storm depends on the hazard classification of the reservoir. Hazard classifications used by Alberta Environment are given in Table 10 along with typical return periods. The classifications and return periods are set by regulatory authorities in each province.

Once a return period precipitation is selected for the emergency spillway, it is translated into an inflow hydrograph. If the reservoir is small, the emergency spillway is usually sized to safely pass the peak inflow rate. If the reservoir is large, the flood should be routed through the reservoir to take advantage of the storage effects. The size of the emergency spillway for the routed storm will be less than that required to pass the peak flow.

The U.S. Soil Conservation Service has published a graphical procedure for sizing emergency spillways and developing stage discharge curves necessary for detailed routing calculations (U.S. Soil Conservation Service 1968). Barfield, Warner, and Haan (1981) illustrate the procedures.

2.6 SEDIMENTATION PROCESSES IN PONDS

The preceding section outlined a number of design approaches that can be used to size the detention volume in a settling pond. For most of these approaches it is necessary to make some estimate of the volume of sediment that a particular pond is capable of removing from a given inflow. This section will describe some of the fundamentals of sedimentation and present a number of the models that have been used to predict pond sediment removal efficiency.

2.6.1 Fundamentals

2.6.1.1 Physical characteristics of sediment. The most important physical characteristic of sediment is its settling velocity. Particle settling is divided into four types based on concentration and the tendency of particles to hinder one another:

1. Discrete particle (Type 1) refers to settling in low

Table 10. Dam hazard classification and return period storm for emergency spillway design[a].

Hazard Classification	Hazard Description		Spillway Design Flows		
	Loss of Life	Economic Loss	Small[b] Dam	Intermediate[c] Dam	Large[d] Dam
Low	None expected (No permanent structures or habitation downstream)	Minimal (underdeveloped to occasional structures or agriculture)	100 yr	100 yr to 0.5 P.M.F.[e]	0.5 P.M.F. to P.M.F.
Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture industry or structures)	100 yr to 0.5 P.M.F.	0.5 P.M.F. to P.M.F.	P.M.F.
High	More than a few (Permanent village or development)	Excessive (Extensive industry or agriculture)	0.5 P.M.F. to P.M.F.	P.M.F.	P.M.F.

[a] Adapted from Alberta Environment (1979).

[b] Storage of 50 to 1000 acre-ft. Dam height less than 40 ft.

[c] Storage of 1000 to 50 000 acre-ft. Dam height between 40 and 100 ft.

[d] Storage greater than 50 000 acre-ft. Dam height greater than 100 ft.

[e] PMF = Probable maximum flood.

concentration solutions in which particles tend to fall independent of each other, i.e., as discrete particles;

2. Flocculant (Type 2) refers to settling in which dilute solutions of particles coalesce together forming particles of larger mass and hence higher settling velocities;
3. Hindered or Zone (Type 3) refers to settling in which particles are so concentrated that forces between particles hinder the settling of neighboring particles. This results in uniform settling of all particles and is characterized by a sharp liquid-solid interface at the top of the solid mass.
4. Compression settling (Type 4) refers to settling in which the particles are so concentrated that interparticle bridging forms a stable structure requiring compression for further settling. The compression force results from the weight of additional particles settling on the top of the settling mass.

All four of these settling modes can occur simultaneously in a sediment pond and therefore each settling type is discussed in detail in the following sections.

2.6.1.1.1 Discrete settling (Type 1). A particle falling in low concentration, turbulent free water moves in response to the difference in the submerged weight of the particle and the drag of the fluid on the particle. At steady-state, the forces are in equilibrium as described by Stokes Law, or

$$V_s = \frac{d^2g}{18N} (SG - 1) \tag{28}$$

where V_s is the particle settling velocity in cm/s, SG is the specific gravity of the particles, d is the particle diameter in cm, g is the gravitational constant 981 cm/s^2, and N is the kinematic viscosity of water in cm^2/s. For a specific gravity of 2.65 and a temperature of 20° C, equation 22 becomes

$$V_s = 8560d^2 \tag{29}$$

where V_s is in cm/s and d is in cm. Stokes' law is valid only for spherical particles falling at velocities such that a linear relationship exists between the drag coefficient and velocity (small silt-size particles and smaller). For larger particles, empirical curves such as those in Figure 15 are typically used.

For a more precise description of discrete settling, the following factors must be accounted for:

1. Particle shape -- Particle shape affects the drag coefficient, which in turn affects the fall velocity. This is normally accounted for by classifying particles according to an equivalent spherical fall diameter. Further information is contained in Graf (1971) and Simons and Senturk (1977);
2. Aggregation -- Aggregation effectively alters the specific gravity of soil particles. Since an aggregate contains pore space, its specific gravity is less than that of the primary particles from which it was formed. Although aggregation decreases the specific gravity, the diameter of an aggregated particle is so much greater than its component primary particle that aggregated particles typically have settling velocities higher than dispersed particles. Site specific fall velocity data should be collected if possible;
3. Turbulence -- The effects of turbulence on settling velocity are somewhat mixed. First, turbulence reduces the drag coefficient C_D by changing the boundary layer from laminar to turbulent. For smaller

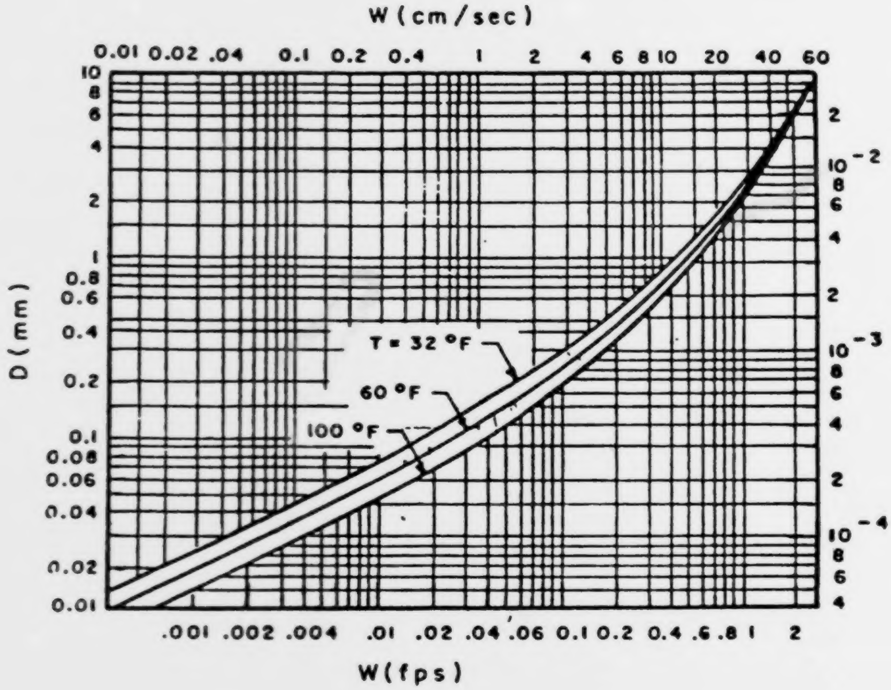


Figure 15. Chart of sedimentation diameter versus fall velocity in quiescent water assuming a specific gravity of 2.65 and spherical shaped particles. (Barfield, Warner, and Haan 1981)

particles, this is offset by the tendency of turbulence to diffuse the particles from a zone of higher concentration near the bottom to a zone of lower concentration near the surface. There are presently no completely acceptable methods for accounting for turbulence. A further discussion is given in Graf (1971); and

4. Flocculation-- Flocculation results in the formation of aggregates through the satisfaction of surface charges on the sediment. The effect is the same as that of aggregation in that the particles formed have a specific gravity less than that of the primary particles. Flocculation will be discussed further in Section 2.6.1.1.2.

Particle settling velocities for Type 1 settling are determined both experimentally and theoretically. The settling velocities of sand sized and larger particles are usually determined by using the size of sieve openings and calculating a settling velocity from Figure 15. For silt and clay size particles, the size distribution must be determined with a settling analysis using a hydrometer or pipette, or by using more recently available electronic instrumentation. If the settling velocity of primary particles is being analyzed, a hydrometer is adequate since the particles are dispersed and the specific gravity is known with a relative degree of confidence. If the settling velocity of a mixture of aggregates and primary particles is being analyzed, it will be necessary to use the pipette method and calculate equivalent fall velocities since the specific gravity will not be known. It should be noted that the sediment is typically composed of a combination of primary and aggregated particles, hence a pipette analysis is preferred. Obviously, a dispersing agent is not used in an aggregate analysis. Details on hydrometer and pipette analysis can be found in standard texts on soil physics and soil mechanics.

2.6.1.1.2 Flocculant settling (Type 2). Colloidal particles in a dilute suspension will sometimes settle as discrete particles and sometimes coalesce and form flocs or numerous particles that have sufficient mass to settle rapidly out of a system. According to Metcalf and Eddy (1979), the degree of flocculation depends on the opportunity for particle contact, the depth of the sedimentation basin, velocity gradients in the basin, the concentration of particles, particle size, and interparticle forces that cause the particles to repel each other. Recent studies associated with surface mine sedimentation indicate that flocculation can be a naturally occurring phenomena, depending on the chemistry of the water and the mineralogy of the clay particles (Tapp, Barfield, and Griffin 1981; Evangelou and Grove 1981). Evangelou et al. (1981) also indicate that dispersion is a possibility under certain conditions. Dispersion will break the aggregated particles down into primary particles, thus decreasing their settling velocity.

A point of beginning in a discussion of both natural and artifical flocculation is a stabilized colloidal suspension. Such a suspension is composed of dispersed or individual surface colloidal particles with a surface charge, separated at considerable distances in an aqueous solution. For clay particles, the surface is negatively charged. The dispersed particles are said to be stabilized since they repel each other. In order for flocculation to occur, the particles must be destabilized and brought together to allow contact and flocculation.

Two theories are used to describe the process of destabilization that may occur naturally or as a result of the addition of an artificial coagulant/flocculant. The first of these is the double electric layer theory, which considers only electrostatic attraction and diffusion. In this theory, the largest concentration of cations (positively charged ions sometimes called counter ions) is near the surface of the colloidal particle because of the electrical attraction of the negative charge on the colloid. The concentration of counter ions decreases with distance away from the colloidal surface as shown schematically in Figure 16. The resulting concentration gradient of positive ions causes a gradient of forces that repel other clay particles that are surrounded by positively charged ions as shown in Figure 17. The layer in which the concentration of counter ions is increased is called the diffuse layer (see Figure 16). As the concentration of ions in the aqueous phase is increased by either natural or artificial means, the thickness of the diffuse layer decreases, causing the magnitude of the repulsive forces to decrease more rapidly at intermediate distances from the surface of the colloidal particle. Thus the forces of repulsion depend on both the solid and aqueous phases. The forces of attraction between colloidal particles are the van de Waals forces, shown schematically in Figure 17, which depend on the mass of the particle. The sum of the forces of repulsion and attraction is used to generate the resultant energy curve. If the concentration of counter ions in the aqueous solution is low, the double layer thickness is high and a potential energy barrier exists, as shown in Figure 17. As the concentration of counter ions increases, the diffuse layer becomes thinner, reducing the forces of repulsion at intermediate distances and ultimately eliminating the potential energy barrier at high concentrations. At this point, the particle is destabilized and flocculation can occur.

In the second destabilization theory, it is proposed that when a synthetic polyelectrolyte is added to the solution, it can attach itself to the surface of a colloidal particle at one or more adsorption sites. Synthetic polyelectrolytes are water-soluble, high-molecular-weight, organic polymers containing chemical groups that undergo electrolytic dissociation in solution resulting in a long chain having highly charged ions (Clark, Viessman, and Hammer 1977). Due to their length, polyelectrolytes will extend beyond the diffuse layer into the solution. This extension can then be attached to vacant adsorption sites on other colloidal particles causing flocculation. A schematic of the processes involved is given in Figure 18. Analytical procedures for predicting the changes in particle size due to flocculation are limited primarily to experimental relationships.

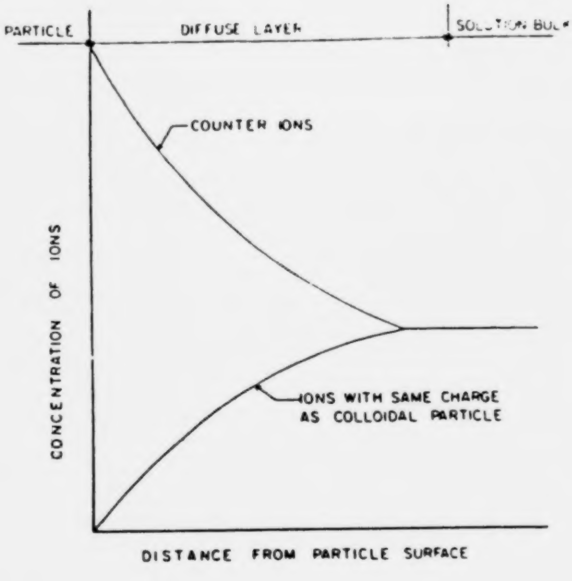


Figure 16. Schematic of electric double layer theory. (Adapted from Tapp, Barfield, and Griffin 1981)

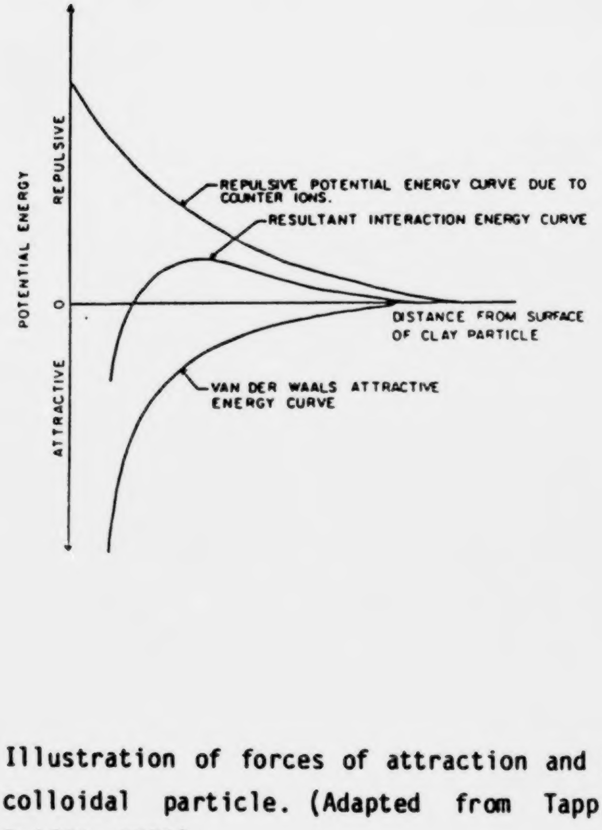

POTENTIAL ENERGY
REPULSIVE
0
ATTRACTIVE
REPULSIVE POTENTIAL ENERGY CURVE DUE TO COUNTER IONS.
RESULTANT INTERACTION ENERGY CURVE
DISTANCE FROM SURFACE OF CLAY PARTICLE
VAN DER WAALS ATTRACTIVE ENERGY CURVE

Figure 17. Illustration of forces of attraction and repulsion near a colloidal particle. (Adapted from Tapp, Barfield, and Griffin 1981)

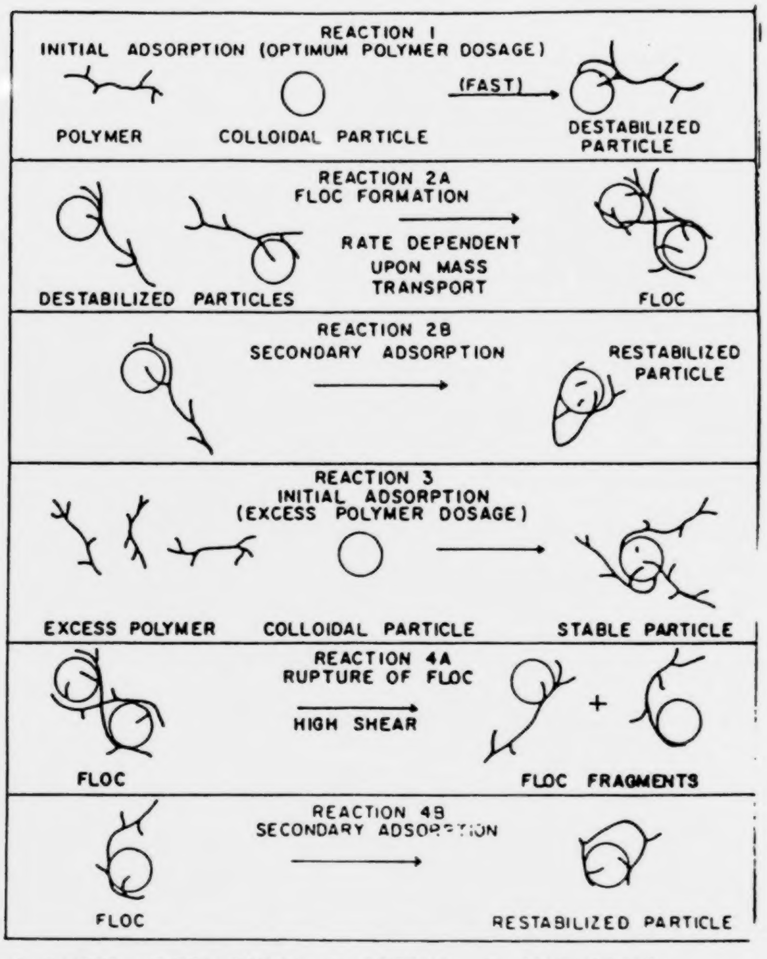

REACTION 1
INITIAL ADSORPTION (OPTIMUM POLYMER DOSAGE)
(FAST)
POLYMER
COLLOIDAL PARTICLE
DESTABILIZED PARTICLE
REACTION 2A
FLOC FORMATION
RATE DEPENDENT UPON MASS TRANSPORT
DESTABILIZED PARTICLES
FLOC
REACTION 2B
SECONDARY ADSORPTION
RESTABILIZED PARTICLE
REACTION 3
INITIAL ADSORPTION
(EXCESS POLYMER DOSAGE)
EXCESS POLYMER
COLLOIDAL PARTICLE
STABLE PARTICLE
REACTION 4A
RUPTURE OF FLOC
HIGH SHEAR
FLOC
+
FLOC FRAGMENTS
REACTION 4B
SECONDARY ADSORPTION
FLOC
RESTABILIZED PARTICLE
1) CHARGE NEUTRALIZATION 2) BRIDGING 3) COMBINATIONS

Figure 18. Illustration of the processes involved in chemical bridging. (Adapted from Tapp, Barfield and Griffin 1981)

Tapp, Barfield, and Griffin (1981) give a summary of the equations developed by various researchers.

A number of artificial flocculants are commonly used to initiate either of the two destabilization mechanisms described previously. Metal salts, such as aluminum sulphate (alum) and ferric sulphate can be used to increase the number of trivalent ions (Al^{3+}, Fe^{3+}) so that the thickness of the diffuse layer is decreased, thereby reducing the forces of interparticle repulsion. However in settling pond applications, synthetic polyelectrolytes have been more widely used than metal salts. Polyelectrolytes that exhibit a positive charge are referred to as cationic polymers, while those with a negative charge are called anionic polymers. Polymers are made up of a number of sub-units, or monomers, that have been reacted to form a larger molecule. The number and orientation of these monomers can be varied to create an almost infinite variety of polymers with different capabilities.

The determination of which polymer is most suitable for a given wastewater is usually based on the results of bench scale jar tests. In a jar test program, one-litre samples of wastewater are dosed with polymer and flocculated under controlled conditions. A large number of samples can be evaluated in a short space of time so that optimum polymer types and dosages can be determined relatively quickly. Jar tests can also be used to quantify the relationship between required polymer dosage and influent TSS, particle size distribution, pH, and temperature. These relationships must be defined before meaningful estimates of the long-term operational costs of a polymer-enhanced pond operation can be estimated.

For situations in which high dosages are required, it is particularly important to evaluate the impact of pH on dosage. Many polymers, especially cationics, are most effective at reduced pH's and if required dosages at neutral pH are high, it may be cost effective to consider an acid feed system. It must be remembered however that the degree of pH reduction that can be considered is often limited by regulatory restrictions on effluent pH.

When establishing optimum polymer dosages it is important to remember that polymers are unlike metal salts in that they are only effective over a relatively narrow range of dosages. Overdosing with a polymer can be just as bad as underdosing. This is illustrated by Figure 19, which shows the impact of dosage on residual turbidity for a number of flocculants. The figure shows that positive ions (Al^{3+}, Ca^{2+}, Na^{+}) do not exhibit an optimum dosage range. Overdosing simply results in an unnecessary expense. However, the polyelectrolyte shown in the bottom graph (hydrolyzed polyacrylamide) cannot be used to excess without a negative impact on effluent quality.

In any discussion of artificial flocculation, questions are inevitably raised regarding the range of incoming particle sizes for which some form of artificial flocculant should be considered. While precise answers for a particular application would require the consideration of a number of site specific factors other than influent particle size distribution, some general observations can be made concerning the likelihood of requiring artificial flocculants in a given situation. In Figure 20, the potential for requiring flocculants in two different types of watershed terrain is rated qualitatively as a function of the minimum influent particle size that makes up any significant proportion of the incoming sediment load. The term 'significant' means proportions in the range of 0.5% to 5% of influent TSS, depending on the magnitude of incoming sediment concentrations (i.e., for a required effluent TSS of 50 mg/L, 0.5% of the minimum incoming particle size can be discharged if the influent TSS is 10 000 mg/L, while 5% can be discharged if the influent TSS is 1000 mg/L).

It must be stressed that the ranges noted in Figure 20 are approximate only and should not be applied to any particular impoundment without considerable caution. The determination of whether or not flocculants are called for in a specific situation should consider site and time specific factors such as local impoundment construction costs, flocculant costs, the cost of supplying power and freshwater, and the magnitude and volume of pond inflows.

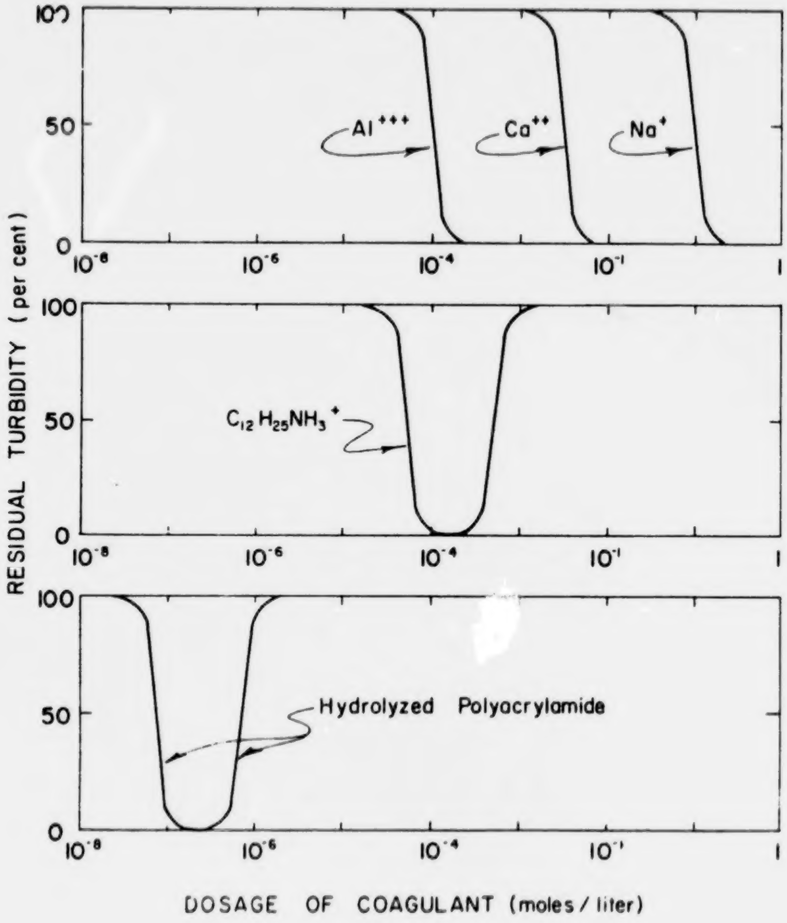


Figure 19. Comparison of coagulant curves for several different coagulants. (Singley 1972)

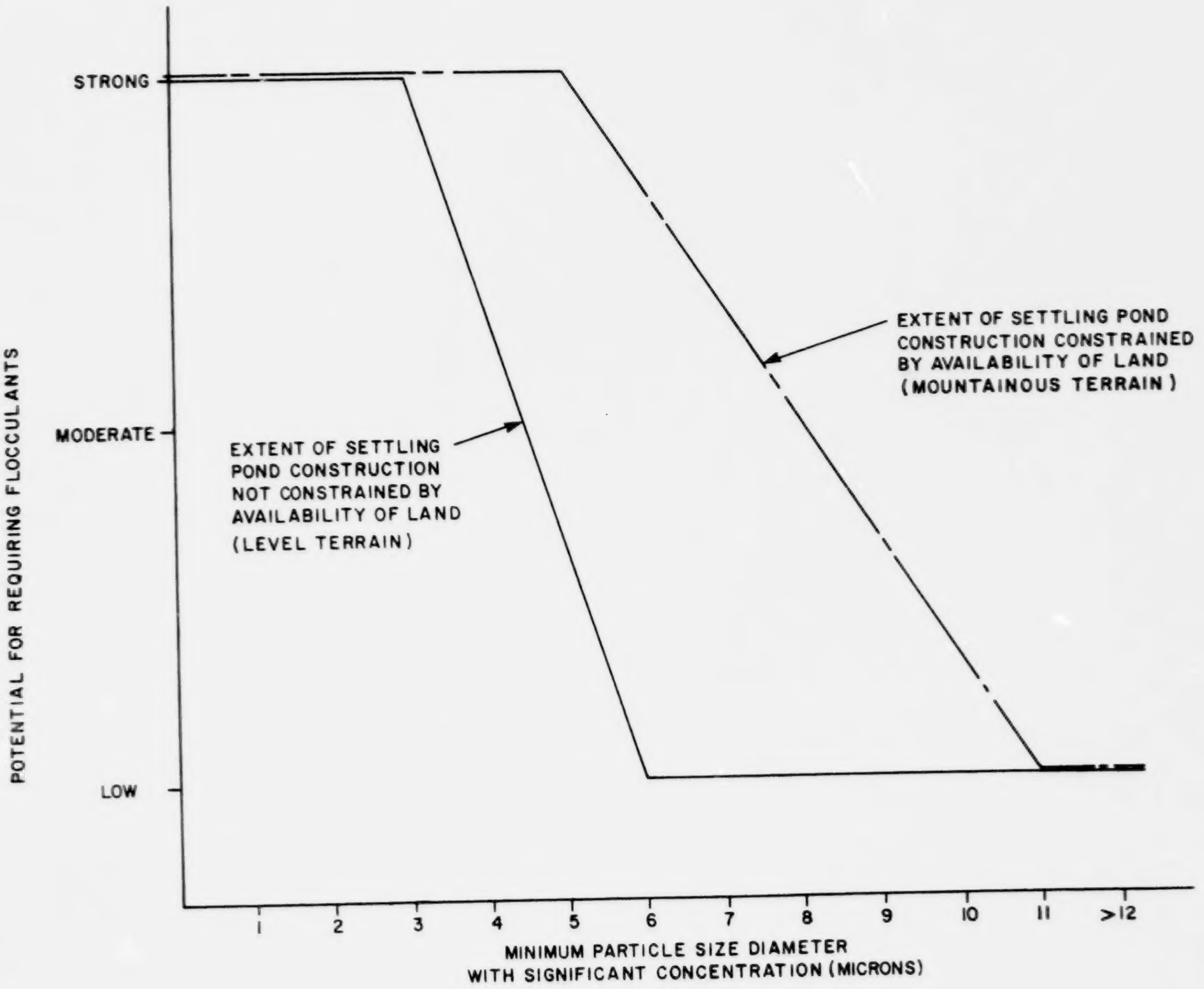


Figure 20. Potential for requiring flocculants for various minimum incoming particle sizes.

2.6.1.1.3 Hindered settling (Type 3). As the concentration of sediment increases numerous particles are in contact with each other. The settling of particles displaces water that must flow upward through the interstices of the contacting particles. As a result, the contacting particles tend to settle as a blanket, with all the particles maintaining the same relative velocity. This is known as hindered settling. Hindered settling would be expected near the bottom of a sediment pond.

The variability encountered with hindered settling is so great that settling tests are typically required for design. Procedures for conducting these tests are described in many references (e.g., Metcalf and Eddy 1979). At the present time no procedures are available for predicting hindered settling on a theoretical basis.

2.6.1.1.4 Compression settling (Type 4). After hindered settling continues for a period of time, a layer of particles with a definite structure begins to form at the bottom of the basin. Further settling is highly variable and usually requires settling tests for design. These settling tests typically are observations of the depth of the liquid-solid interface with time.

2.6.1.1.5 Summary comments on settling. Although it is widely recognized that discrete settling theories do not describe settling velocities that exist under flocculant settling or hindered settling, the standard practice is to either use discrete settling theory or to experimentally measure settling velocities for the particles of interest. If artificial flocculation is to be used to enhance settling, procedures to account for its effect on size distribution must be used.

2.6.1.2 Water chemistry. The casual observer can see that some ponds are relatively clear within a few days after a runoff event and others will not be clear six months later. Recent studies indicate that the chemistry of runoff water is one of the major factors causing this difference (Tapp et al. 1981; Evangelou et al. 1981), primarily due to its influence on flocculation and dispersion. The flocculation or dispersion process can be explained by the double layer theory discussed earlier. As shown in Figure 17, the presence of a potential energy barrier (PEB) will prevent two particles from flocculating. If the ionic strength of cations (concentrations of cations) is increased, the double layer thickness will decrease and the magnitude of the PEB will decrease. At some point, the ionic strength will reach a value such that the barrier is eliminated and the particles will flocculate. Thus, ionic strength has an important effect on flocculation.

In addition to ionic strength, the type of adsorbed cation is also important. Calcium (Ca^{++}) and Magnesium (Mg^{++}) are both divalent ions with a strong electric charge. Thus when adsorbed on the colloidal surface, the charge density quickly goes from negative to positive, resulting in a thin double layer and probable flocculation. Sodium (Na^{+}), on the other hand, is a monovalent ion with a weak charge. In addition, it is highly hydrated (surrounded by water molecules) so that it is weakly adsorbed, thus the charge density does not change as rapidly from negative to positive as in the case of Ca^{++}. The result is the presence of a PEB until very high concentrations of sodium are reached. Therefore, the relative concentrations of Na^{+}, Ca^{++}, and Mg^{++} are also important in the determination of flocculation potential.

Two measures are used to describe the relative magnitude of these ions. One measure, known as the Exchangeable Sodium Percentage (ESP), is the fraction of adsorption sites on the colloidal surface (exchange phase) filled by Na^{+}. The other measure, known as the sodium adsorption ratio (SAR), describes the fraction of sodium ions in solution and can be determined using the following formula:

$$SAR = Na/((Ca+Mg)/2)^{1/2} \tag{30}$$

where Na, Ca, and Mg represent the concentrations of sodium, calcium, and magnesium in meq/L. For soils that are heavy clays, (i.e., high numbers of exchange sites resulting in a high cation exchange capacity), the ESP and SAR are highly related. In soils that are composed of kaolonitic colloids, the number of sites with a double negative charge is not as plentiful as with heavy clays. Na^+ is more easily adsorbed on the kaolonite due to being monovalent and only needing one negatively charged site, thus the SAR and ESP values are typically not equal.

In recent research to predict the presence of flocculation or dispersion, Evangelou et al. (1981) proposed that the thickness of the double layer could be used as a prediction parameter. Using electrical conductivity as a measure of ionic strength, Evangelou et al. analyzed a koalonite soil with a SAR less than 5.0 that had a $Ca^{++} + Mg^{++}$ solution phase. They proposed that the double layer thickness could be estimated from

$$DLT = \frac{1}{[(EC)\ (0.014)]^{0.5}} \qquad (31)$$

where DLT is thickness in arbitrary units and EC is the electroconductivity of the water in mmhos/cm. If the DLT is less than 15, Evangelou et al. (1981) suggest that flocculation will occur. In further studies, Evangelou, Grove, and Barnhisel (1984) found that the DLT value of 15 varied considerably between different spoils.

At the present time, the only conclusive statement that can be made about natural flocculation and dispersion is that it does occur in sediment ponds in response to changes in the mineralogy of the sediments and the chemistry of the water and adsorbed ions. Our ability to predict the exact point of transition from a flocculated to a dispersed state or vice versa is limited.

2.6.1.3 Thermal stratification and density currents. Water has its maximum density at a temperature of 4°C. As a result, in temperate climates, reservoirs have at least two periods during the year when thermal circulation causes complete mixing and considerable turbidity at the pond surface. During the summer months ponds will be warmer and hence, less dense at the surface. In the fall, as cooling at the surface causes temperatures to approach 4°C, the surface layers become more dense and reservoir overturning occurs. As surface temperatures cool below 4°C, the surface becomes less dense and a stable situation develops. In the spring, as the surface warms to 4°C, the process is reversed as the surface again becomes more dense than lower layers. An example of this phenomena is shown in Figure 21. As the temperature gradients become steep, the corresponding density gradients also become steep, making turbulent mixing increasingly difficult. If an eddy or turbulent fluid mass moves from a low to high density area as a result of its random motion, it will find itself in a surrounding that is more dense and will have a tendency to return to its original surroundings. Thus, the density gradient suppresses mixing. Most efforts at modelling the effects of temperature gradients on mixing divide the profile into a mixed and unmixed layer as shown in Figure 22. Mixing occurs as a result of wind, buoyancy forces, and advection.

The presence of density stratification can be important to the performance of a settling pond since the presence of a thermocline (zone of rapid density change with height) represents a barrier to mixing and thus increases dead storage. Models that predict the presence of a thermocline are continuous simulation rather than single storm design oriented models. A continuous simulation model predicts hydrologic events over a long period of time and many storms whereas a single storm model predicts events for a single design storm. The input parameters for continuous simulation models include not only rainfall and runoff, but also temperature, wind speed, long-wave radiation, shortwave radiation, and lake turbidity. At the present time, no simple single storm event models are available that account directly for temperature profile effects.

A companion problem to temperature profile prediction is the presence of density currents. As sediment laden flow moves into a

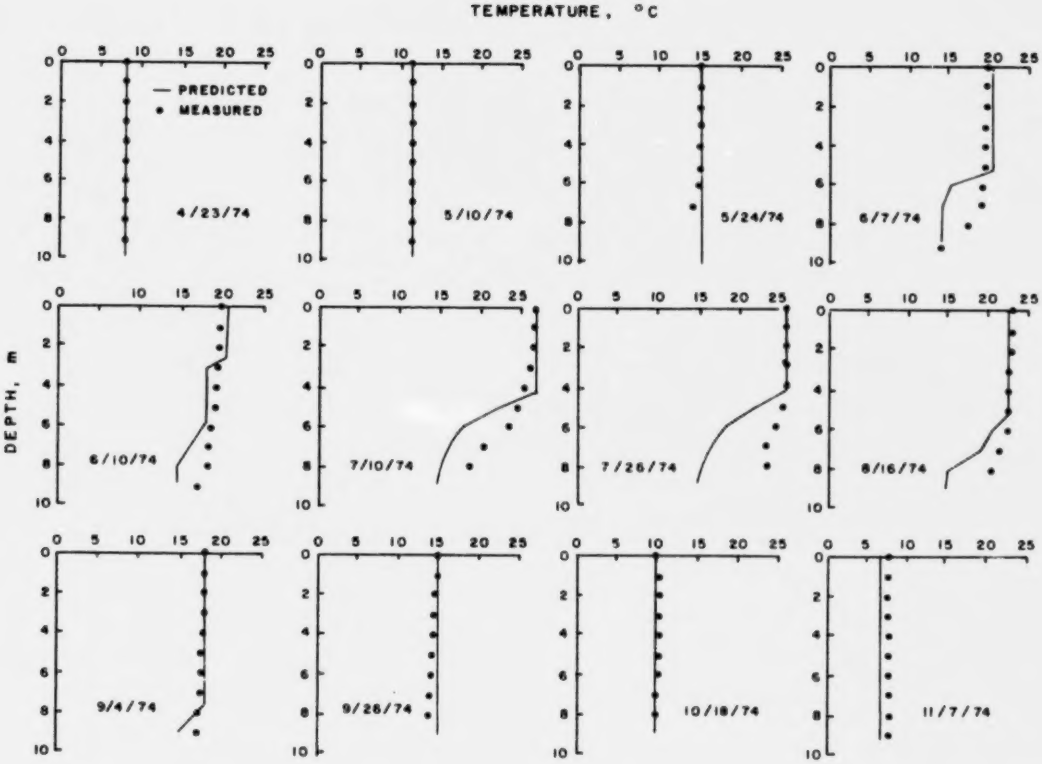


Figure 21. Temperature profiles in Turtle Lake. (Ford and Stefan 1980)

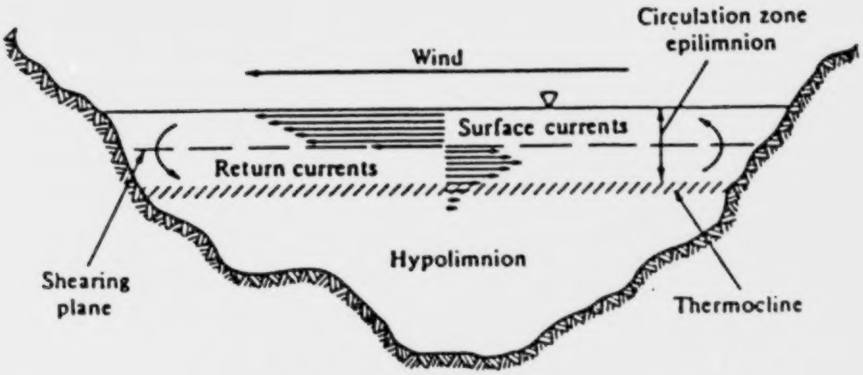

Circulation zone
epilimnion
Wind
Surface currents
Return currents
Hypolimnion
Shearing
plane
Thermocline

Figure 22. Illustration of mixing zones in a pond. (Ford and Stefan 1980)

pond, its density may be different than the fluid in a given layer as a result of its temperature and suspended solids concentration. It would thus tend to plunge into the pond layer with its corresponding density, creating a density current. The location of entry into the pond has a strong influence on pond performance. If the current enters near the bottom of the pond, the sediment will have a smaller distance to fall, but the probable formation of a density current will shorten its detention time since the fluid will bypass much of the pond.

In summary, thermal stratification can generate a thermocline, which is a mixing barrier, effectively causing dead storage volumes to increase. When thermal stratification occurs, density currents can also be present,causing flow to plunge into layers below the surface where the inflow density matches the pond density. Models that predict the phenomena are typically continuous simulation models with complex input requirements.

2.6.2 Sedimentation Models

The sedimentation models that will be discussed in this section vary greatly in sophistication and in their ability to accurately predict sediment removal efficiency. The most effective models are those that maximize the number of significant variables included in the calculations. The variables and factors that control sediment transport through a reservoir are:

1. Physical characteristics of the sediment,
2. Hydraulic characteristics of the basin,
3. Inflow sedimentgraph,
4. Inflow hydrograph,
5. Basin geometry,
6. Chemistry of the water and sediment, and
7. Presence of significant thermal gradients.

In general, predictive techniques for settling pond performance can be divided into either empirical methods or physically based models. Developers of empirically based methods have not attempted to model the internal mechanisms that govern the sedimentation processes, but have simply used observed input and output data to obtain predictive relationships. Empirical methods were popular design techniques until the early 1970's, but are now slowly being replaced by physically based models. In these physically based models, the internal mechanisms that govern hydraulic and/or sediment responses are considered. These mechanisms are typically so complicated that idealization is necessary in order for a tractable solution to be found. For the purposes of this review, physically based models are classified as to the approach used to model the hydraulic component.

The sedimentation models that will be discussed in this section are summarized in Table 11.

2.6.2.1 Empirical models

2.6.2.1.1 Brune's curve. One of the first empirical studies of trap efficiency was made by Brune and Allen. In their study, the percentage of eroded soil caught in a reservoir was predicted as a function of its capacity-watershed ratio. The capacity-watershed ratio was defined as the reservoir's storage capacity per square mile of drainage area. Brune and Allen used 23 reservoirs from Texas to Ohio to obtain their curve.

Brune (1953) later proposed a different set of trap efficiency curves by using a capacity-inflow ratio instead of the capacity-watershed ratio. The capacity-inflow ratio was defined as the ratio between the reservoir capacity and mean annual inflow volume (i.e., capacity -inflow ratio is dimensionless). The well-known Brune curve, shown in Figure 23, was developed using data from 40 reservoirs. This curve assumes that reservoirs are always at least partially filled, so that even large impoundments (i.e., those with capacity-inflow ratios greater than 1.0) will discharge effluent during an average year. Brune originally developed envelope curves for normal ponded reservoirs, which were later used to reflect characteristics of coarse and fine sediment (Heineman 1978). Brune's data

Table 11. Summary of available pond models.

Model Type	Model Name	Computer Program Available	Incorporated with Hydrologic Driving Model	Reference
1. Empirical Models				
	Brune	No	No	Brune (1953)
	Churchill	No	No	Chen (1975)
	Ward's Equations	No	No	Ward (1979)
2. Physically Based Models				
- Overflow Rate Models	EPA-Constant Overflow Rate Models	No	Yes	Hill (1976)
	Variable Overflow Rate Models	No	Yes	Tapp et al. (1981) Curtis McCuen (1977)
- Reactor Models	DEPOSITS	Yes	Yes	Ward et al. (1977) Wilson et al. (1984)
	CSTRS	Yes	Yes	Wilson et al. (1984) Wilson et al. (1983)
	BASIN	Yes	No	Wilson et al. (1984)
- Other Models	FARMPND	(in preparation)		
	CREAMS	Yes	Yes	Knisel (1980)

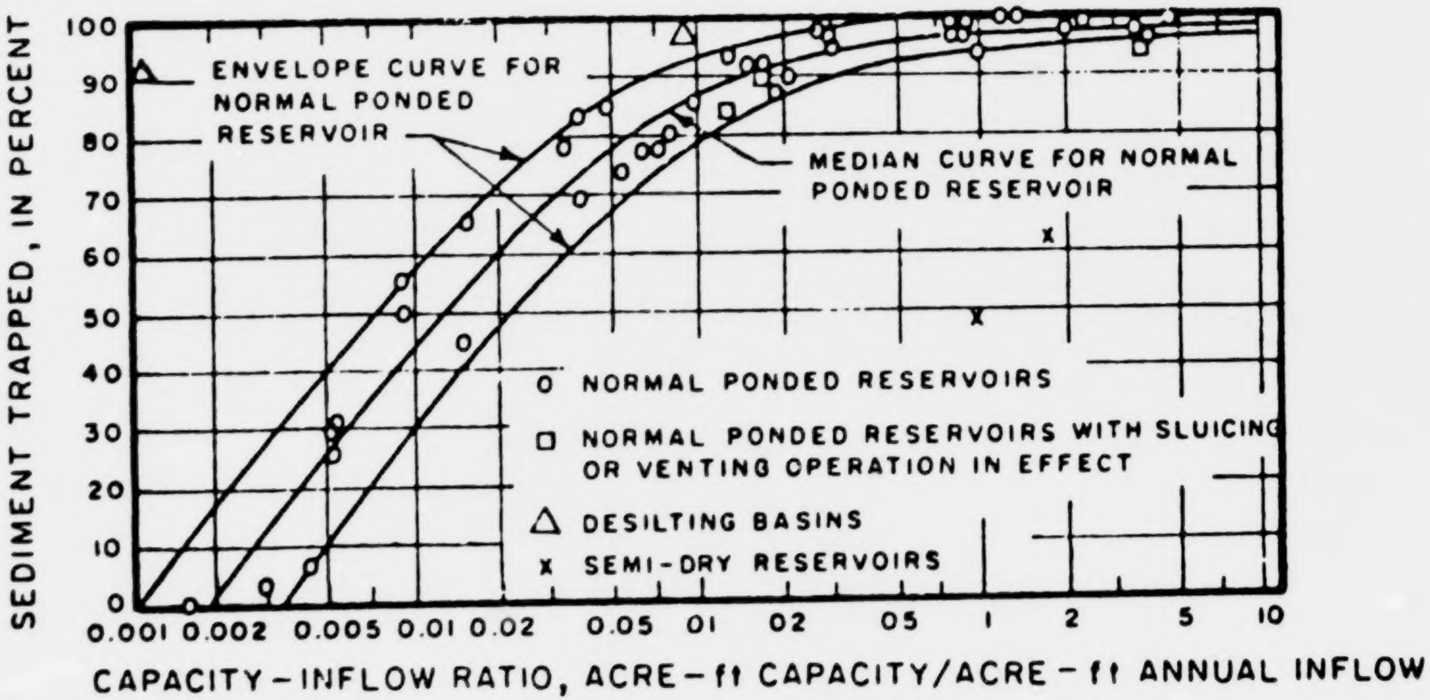


Figure 23. Brune's trap efficiency curves. (Barfield, Warner, and Haan 1981)

set included only two desilting basins (i.e., ponds built primarily for the removal of sediment) and the observed trap efficiency for these points is poorly predicted by Brune's curves. Dendy (1974) examined the use of Brune's curves to predict the trapping efficiency for eleven normally ponded and six dry reservoirs in the southern United States. All but one data point for these reservoirs plotted below Brune's curve. Schiebe (1983) interpreted Brune's curve as a plot of detention time versus trapping efficiency and evaluated the predictions using theoretically derived trapping efficiencies. He concluded that Brune's curves were inadequate for small ponds.

Brune's curve is poorly suited for predicting trap efficiency of single storm events. The capacity-inflow ratio is only a weak indicator of important basin geometry characteristics and hydraulic conditions within the pond. This is especially true for small desilting basins. In addition, Brune's curves do not account for the inflow particle size distribution, a quantity that varies greatly between storms and between watersheds (Ward and Haan 1980; Ward et al. 1981; Rausch and Heinemann 1975). Various studies have shown that the key sedimentology parameters for predicting pond performance are the fall depth, the detention time, and the particle fall velocity. None of these factors was directly considered in the development of Brune's curves.

2.6.2.1.2 Churchill's curve. Churchill (1948) developed curves to predict trap efficiency as a function of a sedimentation index, which he defined as the ratio of retention time to mean flow velocity. The data used for Churchill's curve was primarily obtained from two large Tennessee Valley Authority reservoirs. Churchill's curve is shown in Figure 24.

Churchill's curve has less scatter of points about the line of best fit than Brune's curve, which indicates that it is a better predictor of trap efficiency. Also, the sedimentation index includes the concept of particle residence time, which is a better indicator of the sedimentation process than the capacity-inflow ratio. However,

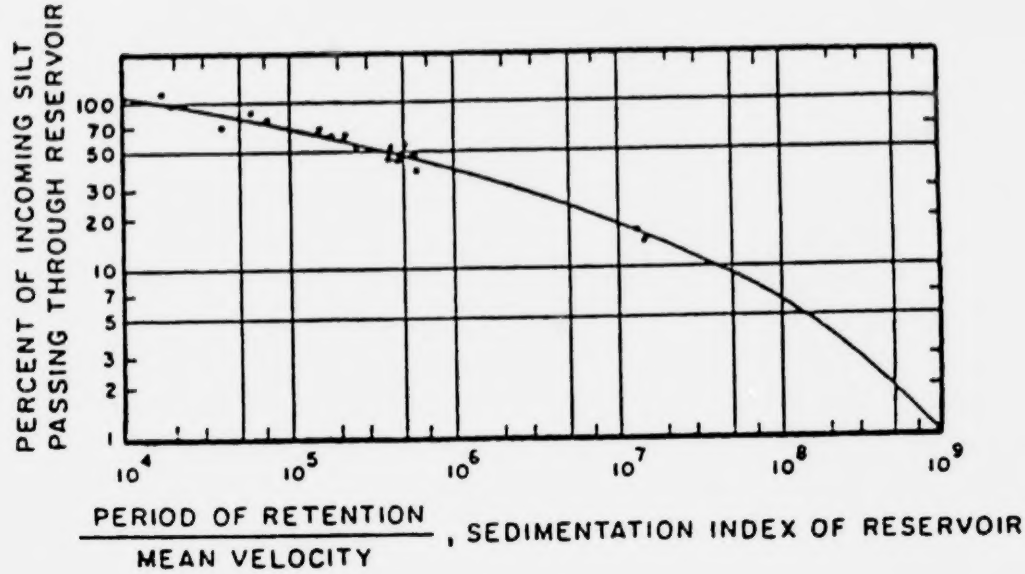


Figure 24. Churchill's curve for determining the percentage of incoming silt passing through a reservoir as related to sedimentation index, TVA reservoirs. (Barfield, Warner, and Haan 1981)

Churchill's method requires that the difficult process of estimating detention time and mean velocity be completed. Furthermore, his method does not directly account for sediment characteristics such as size distribution, hence Churchill's method has many of the same deficiencies as Brune's curve.

With various assumptions on pond geometry and inflow conditions, Chen (1975) rearranged the Brune and Churchill curves so that they could be compared with the overflow rate theory of Camp (1945) (see Section 2.6.2.2). His analysis indicated that both Brune's and Churchill's curves tended to underestimate trap efficiency for coarse material and overestimate trap efficiency for finer sediment. The latter trend was supported by experimental data cited by Tapp, Barfield, and Griffin (1981).

2.6.2.1.3 Ward's Equations. Ward, Haan, and Barfield (1978), Ward (1979), Ward, Haan, and Tapp (1979), and Ward and Haan (1980) presented regression equations that can be used to predict the trap efficiency and peak outflow TSS concentrations for ponds with various input conditions. These regression equations were developed using predicted values of the computer orientated DEPOSITS model (see Section 2.6.2.3). The DEPOSITS model was used because of the large cost associated with gathering data for a wide range of pond geometries and input conditions. In developing the equations the following assumptions were made:

1. The effects of turbulence and short circuiting are insignificant,
2. There is no variation in particle size distribution during a storm event,
3. Water temperature remains constant at 20°C,
4. Inflow sediment concentrations are proportional to the inflow rate, and
5. There is no flocculation within the basin.

Some of the predictive equations that were derived for various outflow conditions are sumarized in Table 12.

Table 12. Ward's regression equations.[a]

Type of Outflow Structure	Sediment Removal Efficiency[b]	Peak Outflow TSS Concentration[b]
1. Perforated Riser		
- Surface layer withdrawal	$E = 102.7+0.000446Q-26.8q_{out}$ $-0.02P_{20}-0.65P_5-4.26D$ $+0.11t_{out}$	-
- Bed layer withdrawal	$E = 100.3+0.000908Q-37.1q_{out}$ $-0.53P_{20}-0.67P_5-3.44D$ $+0.14t_{out}$	-
- Uniform depth withdrawal	$E = 100.6+0.000746Q-31.4q_{out}$ $-0.31P_{20}-0.58P_5-3.28D$ $+0.13t_{out}$	$C_E = 140+3460.5q_{out}+42.0P_5+$ $31.0P_{20}-0.3S-150t_{in}$

continued...

Table 12. Concluded.

Type of Outflow Structure	Sediment Removal Efficiency[b]	Peak Outflow TSS Concentration[b]
2. Drop Inlet Riser	$E = 95.24+1.47(t_{out}-t_{in})-0.65P_5$ $-0.19P_{20}-5.15D_p+0.0006S$ $+ 0.000586Q/q_{out}-1.04t_{out}$	$C_E = 374t_{in}+176.6q_{out}+30.0P_5$ $+45.0P_{20}+0.15S$ $-135t_{out}-194D_p$

[a] Adapted from Haan and Barfield (1978)

[b] where:

E = trap efficiency (%)
Q = inflow volume (m^3)
q_{out} = peak outflow rate (m^3/s)
t_{in} = base time of the inflow hydrograph (hours)
t_{out} = time for 95% of possible withdrawal (hours)

P_5 = percent fines less than 5 microns (%)
P_{20} = percent fines less than 20 microns (%)
D = peak stage at the outlet riser (m)
D_p = peak depth above riser crest (m)
S = permanent storage volume (m^3)
C_E = peak effluent TSS concentration (mg/L)

It is important to note that the relationships shown in Table 12 are empirical equations that should not be projected beyond the range of simulations on which they were based. The equations were originally prepared from data collected for small reservoirs with storm event detention times lower than 10 hours and therefore should be used with caution for large reservoirs and higher detention times. The assumption of a constant water temperature of 20°C also limits their suitability for application to the Alberta foothills and front ranges.

One of the more interesting features of these equations is the fact that they were developed relatively quickly from the outputs of one of the computer models described in subsequent sections. There is no reason that this sort of exercise could not be repeated for any particular area to develop regression equations that would reflect the characteristics and conditions relevant for the region in question.

2.6.2.2 Physically based models: Overflow rate approach

2.6.2.2.1 Steady state flow: Camp's method. The simplest approach to physically modelling sediment basin performance relies on the overflow rate model of Camp (1945). In this approach the settling pond is assumed to behave as an ideal basin that Camp defines as an impoundment with the following characteristics:

1. Steady state and plug flow. For these conditions, the direction of flow is horizontal and the velocity is uniform over all of the basin;
2. A concentration of sediment at the inlet that is completely mixed;
3. Negligible turbulence and quiescent settling. This implies that there will be no resuspension or turbulent diffusion; and
4. A rectangular geometry and constant depth.

A schematic of the ideal basin is shown in Figure 25. The trapping efficiency of an ideal basin can be analyzed using settling

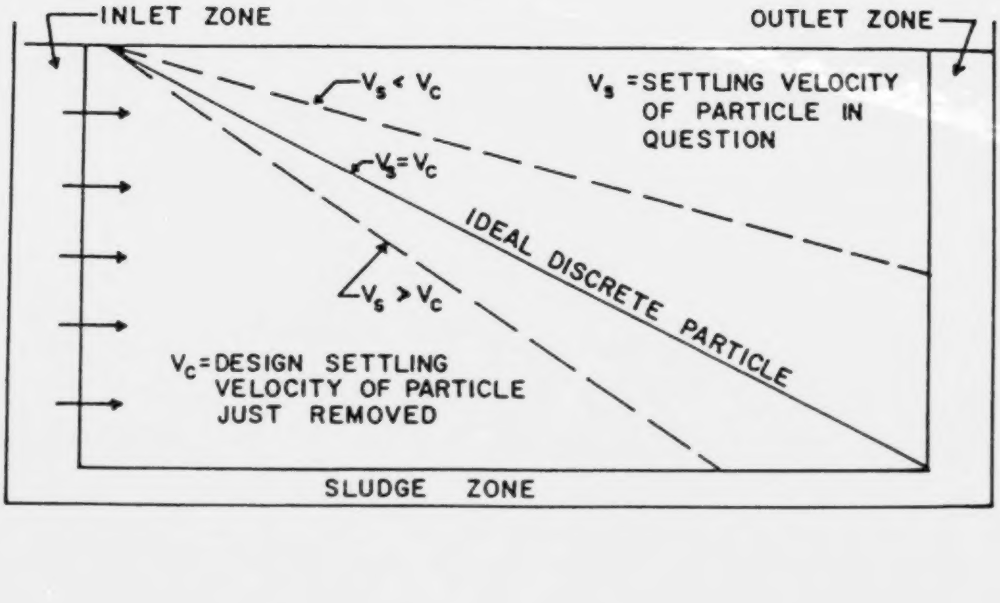

INLET ZONE
OUTLET ZONE
$V_s < V_c$
V_s = SETTLING VELOCITY OF PARTICLE IN QUESTION
$V_s = V_c$
IDEAL DISCRETE PARTICLE
$V_s > V_c$
V_c = DESIGN SETTLING VELOCITY OF PARTICLE JUST REMOVED
SLUDGE ZONE

Figure 25. Illustration of sediment flow trajectories in an ideal rectangular sediment pond. (Barfield, Warner, and Haan 1981)

velocities and an estimate of the influent particle size distribution. Based on a knowledge of the trajectory of a particle through the basin, a critical settling velocity V_c (see Figure 25) can be determined that will just allow a particle to settle to the bottom in its trajectory. Based on geometry, this settling velocity, which is known as the overflow rate, can be shown to be

$$V_c = \frac{Q}{A} \tag{32}$$

where Q is the discharge rate and A is the surface area. For particles with settling velocity V_s (calculated using equation 28) greater than the overflow rate, the fraction trapped is 1.0. For particles with a settling velocity less than V_c, the fraction trapped is

$$F = \frac{V_s}{V_c} \tag{33}$$

If the fraction of particles with settling velocity less than V_c is given by X_c, the total trapping efficiency of a basin can be calculated from

$$E = (1 - X_c) + \int_o^{X_c} \frac{V_s}{V_c}\, dX \tag{34}$$

where X is the fraction finer than a given diameter.

2.6.2.2.2 Steady state flow: USEPA model. The USEPA model is a modification of Camp's ideal settling model in which A is set equal to the basin surface area at the top of the outlet riser and Q is set equal to the peak pond discharge (Barfield, Warner, and Haan 1981). For this method the fraction of particles trapped with a settling velocity V_s is given by

$$F = \frac{V_s A}{1.2\ q_{po}} \qquad (35)$$

where q_{po} is the peak outflow and the factor 1.2 is added to account for non-ideal settling. The calculation procedure for using the EPA model is:

1. The particle size distribution is divided into intervals and the median diameter in each interval is used to compute the settling velocity V_s for that interval;
2. The basin overflow rate V_c is calculated from the routed peak outflow and the effective pond surface area at the permanent pool. The effective area is the area at the permanent pool minus the dead space area;
3. The removal fraction F is calculated for each size interval, and the trapping efficiency E determined using equation 34;
4. The total suspended solids concentration for each time interval of the inflow hydrograph is multiplied by (1-E) to determine the corresponding outflow concentration; and
5. The outflow particle size distribution is calculated based on the fraction of particles removed.

2.6.2.2.3 Unsteady flow: Curtis and McCuen model. Curtis and McCuen (1977) used the overflow rate method of Camp (1945) to predict the performance of detention ponds for nonsteady state conditions. They proposed that the fraction of sediment removed could be calculated using

$$F(t) = (1 - FF_c(t)) + \int_o^{FF_c(t)} \frac{V_s}{V_c(t)} dFF \tag{36}$$

where F(t) is the fraction of particles removed for a plug of inflow corresponding to time t; V_s is the fall velocity corresponding to fraction finer FF; FF_c is the critical fraction finer corresponding to a particle that would fall the entire fall distance; and $V_c(t)$ is the critical settling velocity defined as the pond depth divided by its detention time. Since the detention time and fall depth are unsteady, the value of $V_c(t)$ has to be calculated for each inflow plug. The detention time is calculated by dividing the pond's volume by its volumetric flowrate. Curtis and McCuen (1977) incorporated this pond model into a general watershed hydrology and sedimentology model and demonstrated the usefulness of their model to evaluate alternative strategies in controlling urban runoff.

2.6.2.2.4 Unsteady flow: Tapp's methods. Tapp, Barfield, and Griffin (1981) proposed two alternative methods to improve the USEPA conceptual model. Instead of using a constant (1-E), Tapp's method allows this value to change with each time increment. In Tapp's Method 1, the overflow rate is calculated as

$$(V_c)_i = \frac{K_{or}Q_{oi}}{A_i - K_A A_i} \tag{37}$$

where $(V_c)i$ is the assumed basin overflow rate at a given time, K_{or} is a constant, A_i is the basin surface area at a given time, Q_{oi} is the basin outflow rate corresponding to A_i, and K_A is the fraction of surface area that does not contribute to settling.

Equation 37 was originally developed for a basin with vertical sidewalls. This is rarely applicable to a settling pond and therefore Tapp, Barfield, and Griffin (1981) proposed a second method in which V_c is calculated as a function of fall depth. In Tapp's method 2 the overflow rate is estimated as

$$(V_c)_i = \frac{K_{or} Q_{oi} H_i}{(V_{i-} K_v V_i)} \tag{38}$$

where the overflow rate V_{ci} is a function of time increment i, K_{or} is a constant, Q_{oi}, H_i, V_i are the outflow rate, settling depth, and pond volume at time i, and K_v is the dead space fraction.

The computational steps used in the Tapp Method 1 and the Tapp Method 2 were summarized by Tapp, Barfield, and Griffin (1981) as:

1. The particle size distribution is divided into size intervals and a median diameter is calculated for each interval;
2. The settling velocity V_s, for this median diameter is calculated;
3. For each time on the outflow hydrograph, an effective basin surface area corresponding to the outflow stage is calculated;
4. For each outflow time, a removal ratio F, is calculated for each size interval using equation 37 for Tapp's Method 1 or equation 38 for Tapp's Method 2 and the total trapping percentage E, is calculated using equation 34;
5. The effluent concentration for that time is found by multiplying (1-E) times the corresponding inflow concentration; and
6. The particle size distribution corresponding to that time and outflow concentration is calculated.

Tapp, Barfield, and Griffin (1981) compared predicted effluent concentrations using the USEPA method, the Tapp Method 1 and the Tapp Method 2 to observed concentrations for 28 runs in a pilot scale pond. The 28 runs included chemical flocculation with constant and variable inflow rates as well as quiescent and highly turbulent settling. In general, predictions with the overflow rate models compared poorly with the observed effluent concentrations. It was concluded that the poor fit was due to oversimplification of a complex process.

2.6.2.2.5 Turbulent flow. A theoretical analysis of turbulent flow is much more complex than that required for the quiescent settling described in previous sections due to the presence of an upward diffusion of sediment. Dobbins (1944) developed an analytical solution for the special case in which the turbulent diffusivity is a constant by using what is known as a product series solution. Camp (1944) manipulated Dobbins' solution for the special case in which there is no scour to obtain the results shown in Figure 26. Dobbins verified his equations experimentally using carefully controlled laboratory studies; however, it should be pointed out that Figure 26 applies only to steady state horizontal flow in a rectangular settling tank.

The abscissa on Figure 26 is a form of the Peclet number, given by

$$P_e = \frac{V_s D}{2e} \qquad (39)$$

where D is the depth of the rectangular reservoir and e is the turbulent diffusivity (assumed to be constant). From Figure 26 it can be seen that the effects of turbulence are more pronounced at low Peclet numbers (high e values) and at high values of V_s/V_c. Thus, larger particles are affected most by turbulence and small particles affected the least.

Chen (1975) defined a highly turbulent flow condition as one in which $V_sD/2e$ is equal to 0.01, and determined values of

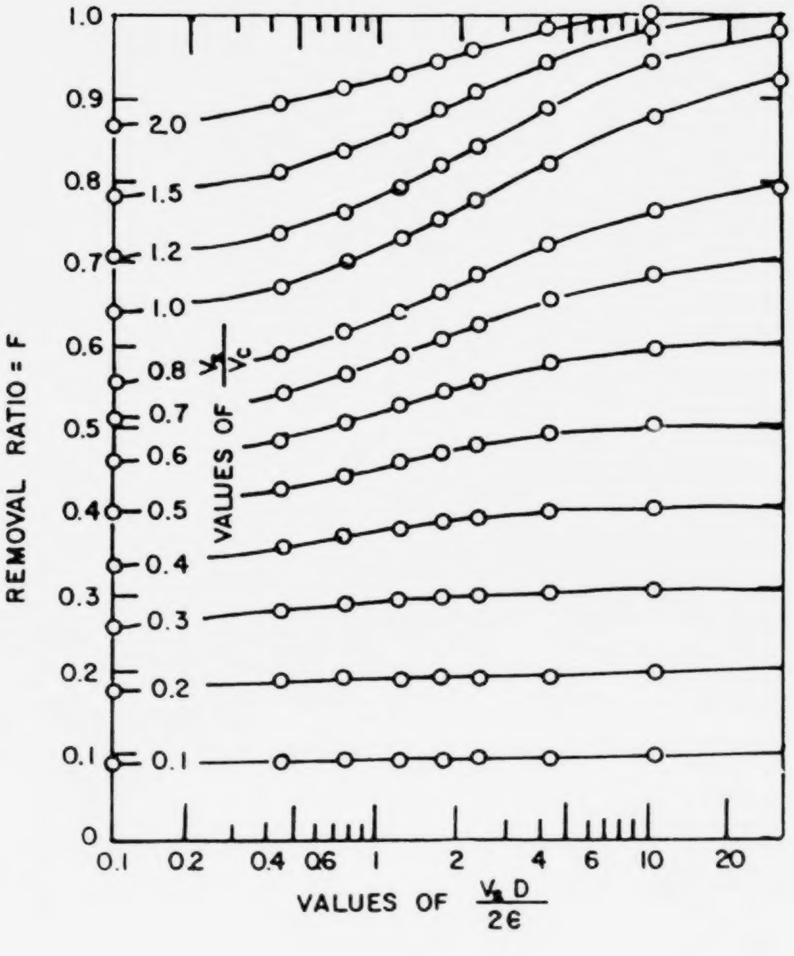


Figure 26. Sediment trapping efficiency of a rectangular reservoir with steady state turbulent flow. (Barfield, Warner, and Haan 1981)

trapping efficiency versus V_sA/Q for that condition. Chen proposed that the turbulent flow values from Figure 26 for Pe = 0.01 could be approximated by

$$F = 1 - \exp \frac{(-V_sA)}{Q} \tag{40}$$

The total trapping efficiency for fully turbulent flow can be found by integrating equation 40 over all size ranges to yield

$$E = 1 - \int_0^1 \exp\left[\frac{-V_sA}{Q}\right] d_x \tag{41}$$

The USEPA and Tapp models described previously can be used to predict sediment removal efficiency for turbulent flow by using a turbulent removal ratio F, calculated with either Figure 26 (Dobbins' and Camp's procedures) or equation 40 (Chen's procedure), in computational procedures similar to those described previously in sections 2.6.2.2.2 and 2.6.2.2.4.

Barfield, Warner, and Haan (1981) compared predictions from the USEPA model, the turbulent model of Chen (1975), and the Dobbins-Camp Model from Figure 26 for a situation with a rectangular basin 1.5 m deep and 3 m wide. The following sediment removal efficiencies were calculated:

Procedure	E
USEPA Quiescent Settling (Equation 29)	0.823
Dobbins-Camp (Figure 19)	0.794
Chen (Equation 33)	0.789

These results would seem to indicate that turbulence is not a particularly important parameter in settling pond design. Such a generalization cannot readily be made. The most pronounced difference between quiescent and turbulent settling occurs at a V_s/V_c value equal to 1.0 as shown in Figure 26. If a large percentage of the particles in a particular influent has diameters with settling velocities near the overflow rate, differences greater than those noted above would be observed. One can conclude then that the effects of turbulence on trap efficiency depend on the influent particle size distribution as well as basin characteristics.

2.6.2.2.6 Summary comments on the overflow rate approach. The overflow rate models are an attractive approach to sediment pond modelling due to their simplicity. In addition to the normal parameters required for hydraulically routing water through a sediment pond, one needs only information on inflow size distribution and concentration in order to calculate effluent concentrations. The major deficiencies of these models are a lack of a theoretical basis for the overflow rate as a function of time and the inability to consider the mixing that inevitably occurs in a pond. These inadequacies limit the suitability of overflow rate models to those situations in which ponds have constant or steady state inflows. This would be the case for impoundments with a pumped inflow that is reasonably constant and protracted during design storm events. However, if the total volume of pumped inflow during a storm event is small relative to the total basin volume, the pond outflow may never reach steady state and overflow rate models will provide poor estimates of sediment removal efficiency.

2.6.2.3 Physically based models: reactor models. Reactor theory can be used to model the hydraulic flow characteristics of a pond so that relatively accurate determinations of particle residence time and mixing can be made. Reactor models are developed by dividing the pond into chambers or reactors in which either plug or completely mixed flow is assumed. A mass balance is performed on each reactor to account for the temporal variation in sediment concentration. One of the major advantages of most contemporary reactor models is that they consider both the inflow and outflow hydrographs to account for the variation in pond detention time during a storm event.

Reactor models represent the present 'state of the art' for practically applicable sedimentation models. The following discussion outlines some of the fundamentals of reactor theory and then presents three of the models that use these basic principles within a format that has practical design applications.

2.6.2.3.1 Single continuous stirred tank reactor. The single reactor that is shown in Figure 27 will be used to introduce reactor theory. For illustrative purposes the inflow reactant will be assumed to be a chemical dye, thus eliminating the deposition problem. Deposition is covered in subsequent sections where the application to ponds is discussed.

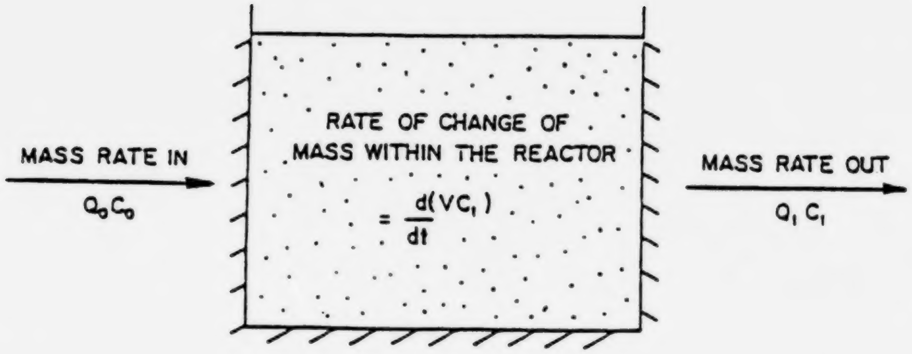
A general mass balance of the reactant in the reactor can be written as

$$\begin{bmatrix}\text{Mass}\\ \text{Rate}\\ \text{In}\end{bmatrix} - \begin{bmatrix}\text{Mass}\\ \text{Rate}\\ \text{Out}\end{bmatrix} = \begin{bmatrix}\text{Rate of Change}\\ \text{of Mass in the}\\ \text{Reactor}\end{bmatrix} \tag{42}$$

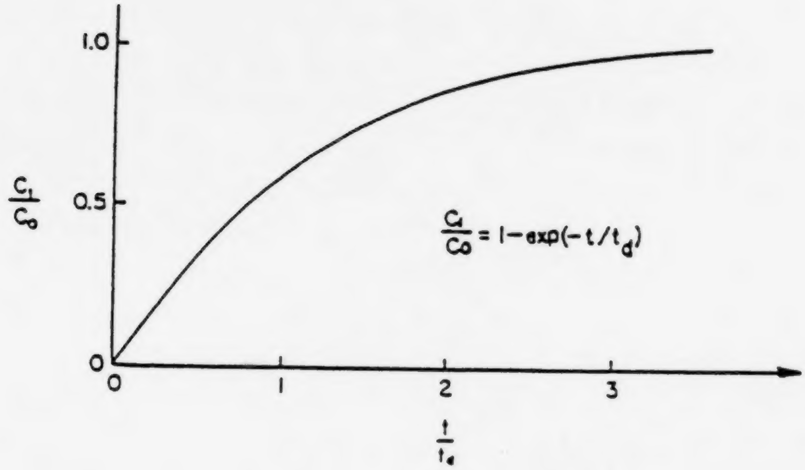
or

$$Q_0C_0 - Q_1C_1 = \frac{d(VC_1)}{dt} \tag{43}$$

where Q_0 and Q_1 represent the volumetric flow rate into and out of the reactor, respectively; C_0 is the influent concentration of the reactant; C_1 is the concentration of the reactant in the reactor; V is


MASS RATE IN
$Q_0 C_0$
RATE OF CHANGE OF
MASS WITHIN THE REACTOR
$= \frac{d(VC_1)}{dt}$
MASS RATE OUT
$Q_1 C_1$

a) SINGLE REACTOR MASS BALANCE



b) OUTFLOW CONCENTRATION FOR A SINGLE REACTOR WITH CONTINUOUS INFLUENT CONCENTRATION

Figure 27. A single continuous stirred reactor. (Wilson, Barfield, and Moore 1982)

the volume of the reactor; and t is time. The solution to equation 43 for steady flow conditions is given by

$$\frac{C_1}{C_o} = 1 - e^{-t/t_d} \tag{44}$$

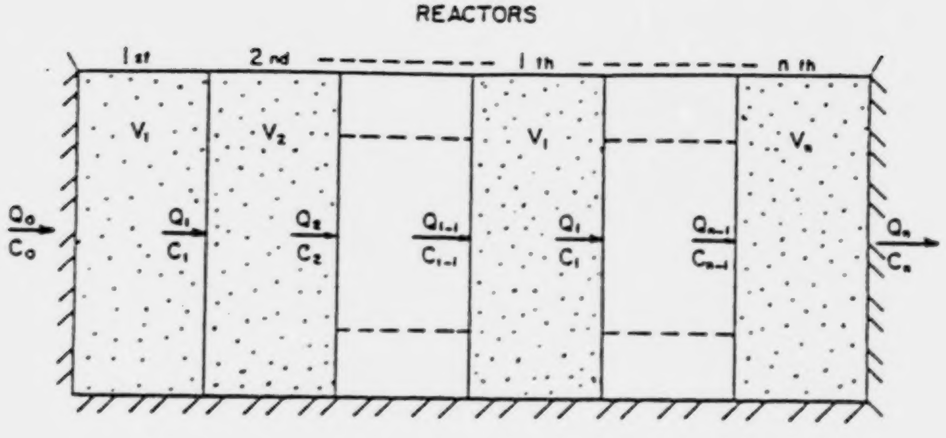
where

$$t_d = \frac{V}{Q_0} \tag{45}$$

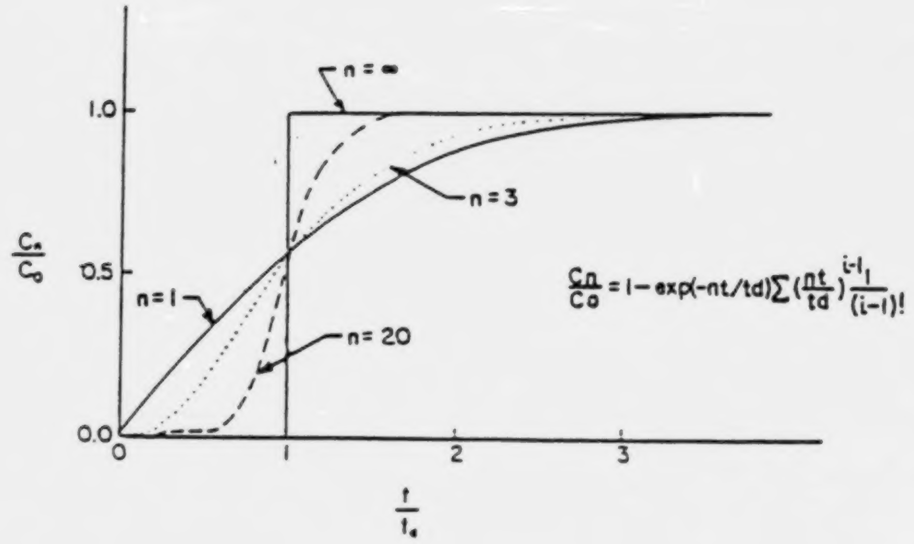
A graphical solution to Equation 44 is shown in Figure 27.

2.6.2.3.2 Series of continuous stirred tank reactors. In the single stirred tank reactor model it is assumed that the inflow concentration is instantaneously mixed with the fluid contained in the pond. Obviously, for a large pond, a time lag is required for the inflow reactant to reach the outlet of the pond. To account for this partial mixing the pond volume is divided into a series of reactors as shown in Figure 28. The influent concentration into a given reactor is assumed to be equal to the effluent concentration of the previous reactor. Because the effluent of a reactor is initially lower than the inflow concentration (see Figure 27), the time lag of the reactant movement between the inlet and outlet of the pond can be considered. Furthermore, since the mixing volume for a particular reactor is reduced when a series of reactors is considered, the dilution effect of that reactor is also reduced.

To develop a solution for a series of continuous stirred tank reactors (hereafter referred to as CSTRS), a mass balance for each reactor is used. For a pond volume that is divided equally among all reactors, the volume in a given reactor V_i is calculated as


REACTORS
1st
2nd
i th
n th
V_1
V_2
V_i
V_n
Q_0
C_0
Q_1
C_1
Q_2
C_2
Q_{i-1}
C_{i-1}
Q_i
C_i
Q_{n-1}
C_{n-1}
Q_n
C_n

a) N CONTINUOUS STIRRED REACTORS



b) OUTFLOW CONCENTRATION FOR N SERIES OF CSTR

Figure 28. A series of continuous stirred reactors. (Wilson, Barfield, and Moore 1982)

$$V_i = V/n \tag{46}$$

where V is the total pond volume and n is the number of reactors. The general solution for n reactors can be derived from

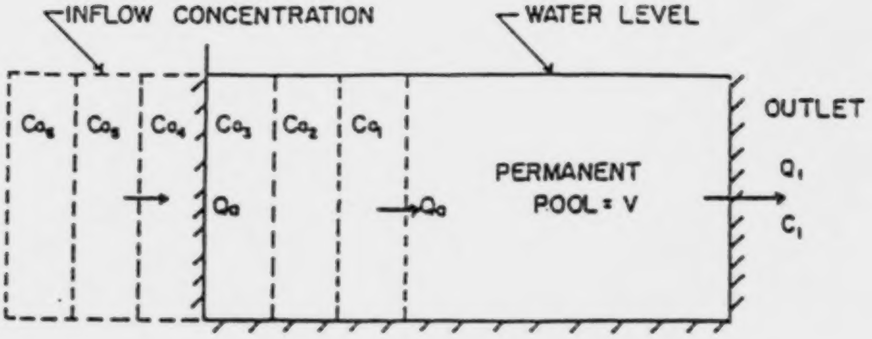
$$\frac{C_n}{C_o} = 1 - \left[\exp\left[-\frac{nt}{f_2 t_d}\right] \sum_{i=1}^{n} \left[\frac{nt}{f_2 t_d}\right]^{i-1} \frac{1}{(i-1)!}\right] \tag{47}$$

where zero factorial is defined as one.

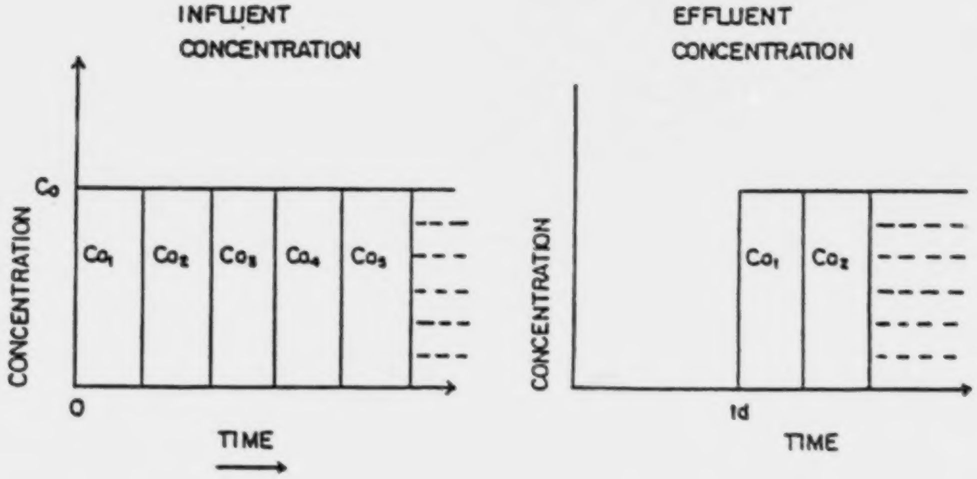
The predicted effluent concentration of a pond for different numbers of CSTRS is shown in Figure 28. As the number of reactors increases, the time delay for the reactant to reach the outlet also increases. An application of the CSTRS theory to pond models is discussed in Section 2.6.2.3.7.

2.6.2.3.3 Plug flow reactor. As the number of CSTRS approaches infinity, the hydraulic response of a pond approaches a plug flow reactor (see Figure 28). In a plug flow reactor the volume of a particular mixing chamber, as defined by Equation 46, approaches zero. Consequently, the inflow concentration is not mixed with any portion of the current pond volume. If a dye tracer is used as the reactant (i.e., deposition does not occur), the effluent concentration will equal the influent concentration except for the travel time between the inlet and the outlet of the pond.

A conceptualized view of a plug flow reactor is shown in Figure 29. Steady state conditions are again assumed and the inflow concentration is broken into plugs of concentration, C_0. Each plug proceeds through the pond without any mixing with the permanent pool or other plugs. The first plug will reach the outlet of the pond when the permanent pool is completely displaced. The time required to displace the permanent pool is equal to the theoretical detention time (i.e., $t_d = V/Q_0$).


INFLOW CONCENTRATION
WATER LEVEL
OUTLET
Co6
Co5
Co4
Co3
Co2
Co1
Qo
Qo
PERMANENT
POOL = V
Q1
C1

a) CONCEPTUALIZED VIEW OF A PLUG FLOW REACTOR



b) INFLUENT CONCENTRATION AND RESULTING EFFLUENT CONCENTRATION FOR A PLUG FLOW REACTOR

Figure 29. Schematic of a plug flow reactor. (Wilson, Barfield, and Moore 1982)

Plots of the influent and effluent concentrations are shown in Figure 29 for the conceptualized plug flow reactor. The effluent concentration is equal to the concentration of the permanent pool (assumed equal to zero) until after the theoretical detention time. The effluent concentration is then equal to the concentration of the influent. For a plug flow reactor, the first inflow plug is also the first outflow plug, the second inflow plug is the second plug discharged, and so on. Mathematically, the effluent concentration is given by

$$C_1\ (t) = 0 \text{ if } t < t_d \qquad (48)$$
$$C_1\ (t) = C_0\ (t) \text{ if } t > t_d$$

An application of the plug flow model to ponds is discussed in Section 2.6.2.3.6.

2.6.2.3.4 Short-circuiting and dead space. To illustrate the concepts of short-circuiting and dead storage space, the single CSTRS shown in Figure 30 will be used. Short-circuiting is the fraction of the inflow that bypasses the reactor. In Figure 30 short-circuiting is equal to $(1-f_1)\ Q_0$, where f_1 is the fraction of discharge that enters the pond and Q_0 is the steady state inflow discharge. The short-circuited flow is assumed to instantaneously arrive at the outlet of the pond.

Dead space is the fraction of a reactor volume that does not mix with the influent concentration. In Figure 30 dead space is equal to $(1-f_2)\ V$, where f_2 is the fraction of the reactor volume that is active and V is the total volume of the reactor. To summarize, short-circuiting is the portion of the flow that bypasses the CSTR, while dead space is the portion of the reactor volume that does not mix with the inflow concentration. In the real world, short circuiting as conceptualized in Figure 30 does not exist, and therefore most approaches to reactor modelling ignore short-circuiting and lump all non - ideal behaviour into the dead storage

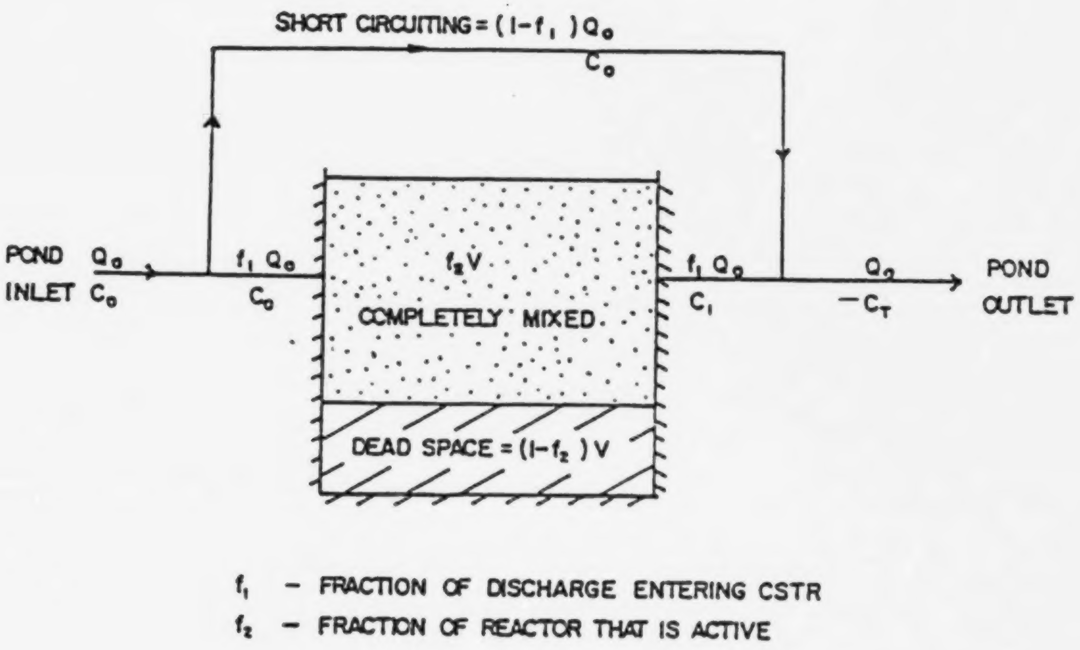

SHORT CIRCUITING = $(1-f_1)Q_o$
C_o
POND INLET
Q_o
C_o
$f_1 Q_o$
C_o
$f_2 V$
COMPLETELY MIXED
DEAD SPACE = $(1-f_2)V$
$f_1 Q_o$
C_1
Q_o
$-C_T$
POND OUTLET
f_1 – FRACTION OF DISCHARGE ENTERING CSTR
f_2 – FRACTION OF REACTOR THAT IS ACTIVE

Figure 30. A single CSTR with short-circuiting and dead space. (Wilson, Barfield, and Moore 1982)

term. For the CSTRS model, equation 47 as modified to account for dead storage becomes

$$\frac{C_n}{C_o} = 1 - \left[\exp\left[\frac{-nt}{f_2 t_d}\right] \sum_{i=1}^{n} \left(\frac{nt}{f_2 t_d}\right)^{i-1} \frac{1}{(i-1)!} \right] \qquad (49)$$

where f_2 is the fraction of the pond that is active and is given by

$$f_2 = 1 - f_3 \qquad (50)$$

where f_3 is the fraction of the pond that is dead storage.

2.6.2.3.5 Estimating the hydraulic response of a pond. Once the designer has selected one of the preceding reactor models, assumptions must be made regarding the percentage of dead space and, for CSTR models, the number of reactors that the basin is to be divided into. For the evaluation of an existing pond this can be done using the results of dye tracer studies. In dye studies a suitable chemical dye is injected at the inlet of a pond and the effluent concentration of this dye is monitored at the pond outlet. The inflow conditions (e.g., flow conditions and dye concentration) for the basin must be controlled to satisfy the assumptions used to develop the different reactor models. The results of the dye studies are then compared with the predicted effluent concentration versus time relationships derived from a variation of Equation 49 for different combinations of dead space and number of reactors (CSTRS model only). The configuration that most closely matches the observed data defines the reactor parameters that will be used to estimate sediment removal efficiency. A more detailed discussion of the procedure for establishing reactor parameters is provided by Wilson, Barfield, and Moore (1982) and Griffin, Barfield, and Warner (1983).

Reactor parameters for the design of new ponds are more difficult to establish accurately. When using plug flow models the appropriate percentage of dead space can be approximated as a function of the pond length to width ratio. Griffin, Barfield, and Warner (1983) found that dead storage averaged 15% of the total storage for 'long' ponds and about 25% for 'short' ponds. They suggested that the breakpoint between 'long' and 'short' ponds be established at a length to width ratio of 2:1, when the pond length to width ratio is defined as the length of the average flow path from the pond inlet to outlet over the average width of the flow path.

For CSTRS models Griffin, Barfield, and Warner (1983) found that the relationship between dead storage and pond length to width ratio was essentially the same as that noted above for plug flow models. They also determined that the best fit with tracer dye results was usually obtained when using two CSTRS in series. Griffin, Barfield, and Warner (1983) found that this optimum number of CSTRS, as well as the dead storage versus length to width ratio relationship, were not particularly sensitive to a dimensionless momentum factor that they defined as

$$(p_i Q_i U_i / pqV) \times 10^5 \tag{51}$$

where pi is the density of the inflow water, Q_i is the inflow flow rate, U_i is the velocity of the inflow, q is the density of water in the pond, g is the gravitational constant, and V is the pond volume.

One important comment should be made on the use of reactor models to estimate dead storage. Reactor models are idealizations of physical processes and as such do not accurately represent all of the phenomena occurring in a reactor. For example, the models treat dead storage as a stagnant area that does not interact in the sedimentation process. In reality, fluid does eventually diffuse out of these dead storage regions. Thus, regions designated as dead storage are not totally ineffective in the sedimentation process, but are simply less effective than other pond volumes. Dead storage, therefore, is simply the parameter that allows the reactor model to fit the observed dye tracer data.

2.6.2.3.6 The DEPOSITS plug flow reactor model. Ward, Haan, and Barfield (1977) developed a comprehensive reactor model called DEPOSITS, an acronym for Detention Performance Of Sediment in Trap Structures. The DEPOSITS model is a computer program that was developed to directly consider the following factors in the performance of detention ponds:

1. Inflow sedimentgraphs and hydrographs;
2. Hydraulic and geometric characteristics of a basin;
3. Particle size distribution of sediment; and
4. Viscosity of the flow.

The flow in DEPOSITS is idealized as plug flow and the detention time, portion of inflow sediment mass, and average water depth are determined for each plug. The water depth is divided into four layers and the sediment removal efficiency as well as effluent concentrations are determined by calculating the amount of sediment which settles out of each layer, with settling velocities calculated using Stoke's law. DEPOSITS is written so that short-circuiting and nonideal settling can be accounted for by the use of empirical factors. The effects of permanent pools, location of spillways, and density currents are considered, however the impact of thermal stratification is not. The input data requirements for the DEPOSITS model are summarized in Section 2.6.2.6.

DEPOSITS has frequently been used as a design tool for evaluating the effectiveness of detention ponds constructed for surface mining operations. Wilson, Barfield, and Moore (1982) modified DEPOSITS and incorporated it as one of the computational algorithms in SEDIMOT II, a general hydrology and sedimentology simulation model (see Section 2.7). A slight modification of DEPOSITS was also used by Smith (1981) in his Watershed Sediment Yield Model.

The computational algorithm in the DEPOSITS model is segmented into three steps:

1. Pond geometry characteristics,
2. Hydrograph routing procedures, and
3. Sedimentgraph routing procedures.

The pond geometry portion of DEPOSITS calculates the characteristics of a pond that are a function only of stage height. For each stage point that is entered by the user, the storage volume, average pond depth, and storage characteristic curves are determined. These values depend only on the pond geometry and spillway characteristics and are thus independent of the inflow event. One of the more important of the above parameters is the average depth a particle must fall to be trapped. Since the water depth varies from the inlet to the outlet of a pond, the fall depth of particles also varies. To simplify, the DEPOSITS model uses a spatially averaged depth that is defined as

$$D(S_j) = \frac{\sum_{i=1}^{N} D_i^2 (A_i - A_{i-1})}{\sum_{i=2}^{N} D_i (A_i - A_{i-1})} \tag{52}$$

and

$$D_i = S_j - (S_i + S_{i-1})/2 \tag{53}$$

where $D(S_j)$ is the volume weighted average depth corresponding to stage height S_j, and A_i is the area corresponding to stage height S_i.

The hydrograph routing portion of DEPOSITS is used to determine the outflow hydrograph and to store inflow and outflow results that are needed subsequently to estimate the trap efficiency of the basin. These results include:

1. The relationship between inflow sediment load and inflow volume,

2. A tabulation of pond depths,
3. A summation of outflow volume, and
4. The centre of mass of the inflow hydrograph.

The outflow hydrograph is calculated using the Puls' reservoir routing method (see Barfield, Warner, and Haan 1981). The stage-discharge relationship that is required to complete this calculation is either input manually by the user or, if a drop inlet spillway is used, calculated internally by the program.

In the DEPOSITS model the sediment is routed through the pond by assuming plug flow. By making this assumption, a particular plug of outflow can be traced to its entrance into the pond from the inflow hydrograph. This allows the initial sediment mass, detention time, and average pond depth to be determined for the plug in question. With this information and an initial particle size distribution, the amount of mass that is trapped as the sediment moves from the pond's inlet to its outlet can be calculated.

Each plug in the pond's flow is subdivided horizontally into four layers of equal depth. The sediment remaining in suspension within each layer is calculated as a function of time using a version of Stokes' law modified to account for the non-spherical shape of clay and colloidal particles. The percentage of the total outflow discharge associated with each plug is calculated and the fraction of the initial sediment content that is removed by each plug is determined. The sum of these incremental sediment discharges provides an estimate of the total mass of sediment removed during a storm event.

Ward, Haan, and Barfield (1977) compared the predicted trap efficiency from DEPOSITS to observed values for ponds monitored by the USEPA. Although the input parameters were not always well defined, there was excellent agreement between predicted and observed trap efficiency. Additional verification studies on detention ponds for strip mine operations were conducted by Ward and Haan (1980). Only a limited number of events were successfully monitored; nevertheless, predictions of trap efficiency with the DEPOSITS model were quite good. Ward et al. (1981) also compared the predicted trap efficiency to observed trap efficiency for two large agricultural reservoirs. Although the DEPOSITS predictions of the timing of the outflow sedimentgraph were not accurate, observed trap efficiencies were well predicted. Tapp, Barfield, and Griffin (1981) compared the predictive ability of DEPOSITS to observed data gathered in a pilot-scale sedimentation pond and concluded that the effluent sedimentgraphs were poorly predicted by DEPOSITS. Subsequent evaluations by Wilson et al. (1981) with Tapp, Barfield, and Griffin (1981) data sets also indicated that the effluent sedimentgraph was not well predicted. A summary of the predicted trap efficiencies is given in Table 13 and predicted effluent concentrations in tables 14 and 15.

The above studies demonstrated that the major deficit in the DEPOSITS model is the lack of procedures to account for mixing in a pond, both between plugs of flow and with previously stored flow. As will be shown in the next section, by accounting for this mixing, the timing of peak effluent concentrations is more accurately predicted.

2.6.2.3.7 The CSTRS reactor model. In order to cover the lack of a mixing routine in the DEPOSITS model, Wilson, Barfield, and Warner (1983) and Wilson, Barfield, and Colliver (1984) used the continuous stirred reactor in series theory (CSTRS) discussed earlier to model pond performance. By using CSTRS, both the mixing and delayed flow characteristics of a pond can be modelled. The CSTRS model is similar to the DEPOSITS model in that it uses the same procedures for defining the pond geometry, spillway characteristics, and the outflow hydrograph. Its major difference from DEPOSITS is the manner in which sediment is routed through a pond. The CSTRS model does not consider the effects of thermal stratification. The input data requirements for the model are summarized in Section 2.6.2.6.

In the CSTRS model, mixing between inflow concentrations and previously stored flow is modelled by dividing the pond into a series of reactors of equal volume as shown by Figure 31. A mass

Table 13. Summary of DEPOSITS predicted trap efficiency for field studies.

Pond Number	Storm Inflow (watershed - mm)	Detention Time (hrs)	DEPOSITS Predicted Trap Efficiency (%)	Observed Trap Efficiency (%)
1. USEPA Study (Ward, Haan, and Barfield 1977)				
1	-	-	94	98
2	-	-	95	92
3	-	-	82	91
4	-	-	90	89
2. Calahan Reservoir Study (Ward et al. 1981)				
1	32	70	67	68
	33	57	73	78
	24	93	78	78
	50	35	76	79
	23	100	88	83
	50	24	84	85
3. Bailey Reservoir Study (Ward et al. 1981)				
1	43	295	87	68
	38	358	85	87
	64	225	89	87
	38	359	72	85
	86	81	92	84
4. East Kentucky Surface Mine Study (Ward and Haan 1980)				
1	19	-	68	66

Table 14. Predicted and observed trap efficiencies for DEPOSITS and CSTRS models.[a]

Run No.	Peak Inflow Rate (L/m)	Peak Influent Concentration (mg/L)	Observed Trap Efficiency (%)	Predicted Trap Efficiency	
				DEPOSITS Model (%)	CSTRS Model (%)
18	53	1 330	74	77	78
19	59	19 500	85	83	84
22	24	17 800	88	91	88
24	102	17 800	85	88	89
26	58	154 000		81	80
28	102	76 000	84	74	79

---

[a] Efficiencies based on data from Tapp, Barfield, and Griffin (1981).

Table 15. Predicted and observed peak effluent TSS concentrations for overflow rate, DEPOSITS, and CSTRS models.[a]

Run No.	Observed Peak Effluent TSS (mg/L)	Predicted Peak TSS Concentration		
		Overflow Rate Tapp 2 (mg/L)	DEPOSITS Model (mg/L)	CSTRS Model (mg/L)
18	250	235	330	230
19	2 200	2 250	4 100	2 720
22	990	1 400	1 850	1 100
24	2 700	2 050	3 700	2 000
26	25 000	16 000	35 000	25 500
28	17 000	8 600	24 000	12 750

[a] Calculations based on data from Tapp, Barfield, and Griffin (1981).

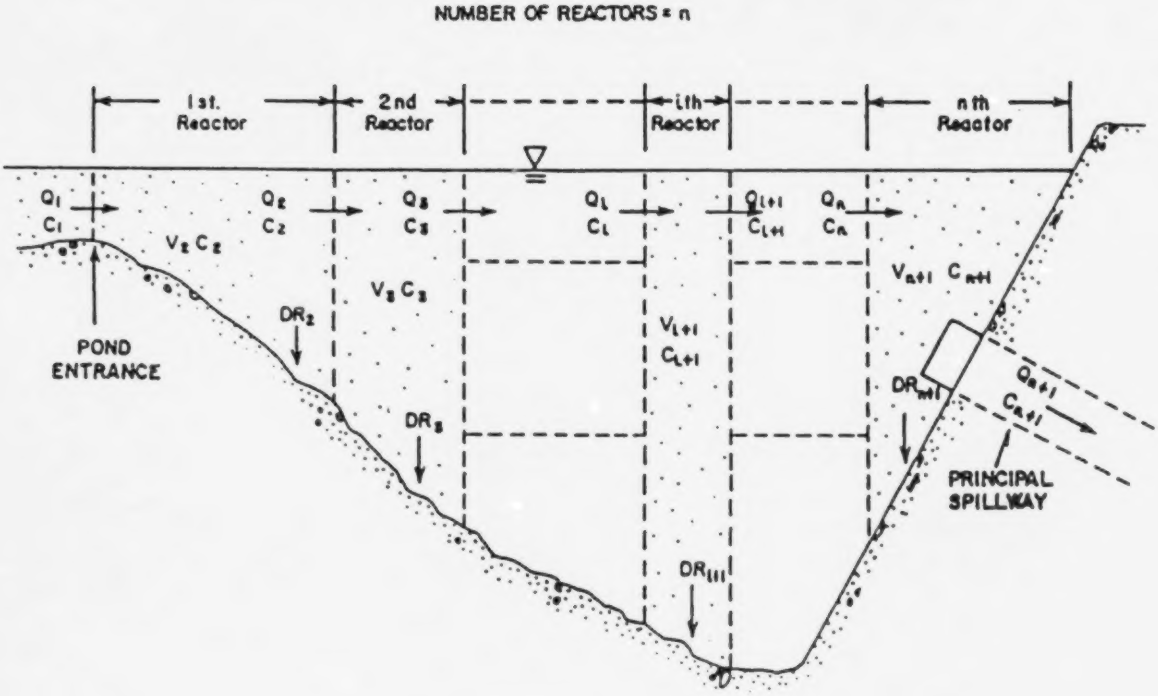

NUMBER OF REACTORS = n
1st. Reactor
2nd Reactor
ith Reactor
nth Reactor
Q_1
C_1
$V_2 C_2$
Q_2
C_2
Q_3
C_3
$V_3 C_3$
Q_i
C_i
Q_{i+1}
C_{i+1}
V_{i+1}
C_{i+1}
Q_n
C_n
V_{n+1} C_{n+1}
DR_2
DR_3
DR_{i+1}
DR_{n+1}
Q_{n+1}
C_{n+1}
POND ENTRANCE
PRINCIPAL SPILLWAY

Figure 31. Pond divided into a series of CSTRS. (Wilson, Barfield, and Moore 1982)

balance is conducted on each reactor to determine its effluent concentration. The pond outflow concentration corresponds to the effluent concentration of the final reactor (i.e., n^{th} reactor in Figure 31). In Figure 31, the inflow and outflow are identified by Q_1 and Q_{n+1} respectively, while the influent concentration and the effluent concentration are identified by C_1 and C_{n+1}, respectively. A mass balance on any arbitrary i^{th} reactor can be written as

$$\begin{bmatrix} \text{Mass Rate} \\ \text{Into} \\ \text{Reactor i} \end{bmatrix} - \begin{bmatrix} \text{Mass Rate} \\ \text{Out of} \\ \text{Reactor i} \end{bmatrix} - \begin{bmatrix} \text{Deposition} \\ \text{Rate in} \\ \text{Reactor i} \end{bmatrix} = \begin{bmatrix} \text{Rate of Change} \\ \text{of Mass in} \\ \text{Reactor i} \end{bmatrix} \qquad (54)$$

For a continuous stirred reactor, the reactor's effluent concentration is equal to the concentration of the fluid contained in the reactor. Hence Equation 54 can be written as

$$Q_i C_i - Q_{i+1} C_{i+1} - DR_{i+1} = \frac{d(V_{i+1} C_{i+1})}{dt} \qquad (55)$$

where Q_i and C_i are the inflow and the influent concentration, respectively; Q_{i+1} and C_{i+1} are the outflow discharge and the effluent reactor concentration, respectively; DR_{i+1} is the deposition rate in the reactor; V_{i+1} is the volume of the reactor; and t is time.

Equation 55 can be solved using a finite difference approximation to yield the following equation for the effluent concentration in a particular reactor at the end of the time increment used to route the sediment flow:

$$C^n_{i+1} = \frac{\Delta t(Q_i C_i - Q_{i+1} C^o_{i+1}/2) - DEP + C^o_{i+1} V^o_{i+1}}{V^n_{i+1} + \Delta t\ Q_{i+1}/2} \qquad (56)$$

where Δt is the time increment used to route the sediment flow; Q_iC_i is the average inflow mass rate; Q_{i+1} is the average outflow discharge rate; V^0_{i+1} and V^n_{i+1} are the reactor volumes at the beginning and end of the time increment, respectively; C^0_{i+1} and C^n_{i+1} are the effluent/reactor concentrations at the beginning and the end of time increment, respectively; and DEP is the mass of sediment deposited within Δt. In Equation 56, Q_iC_i is defined from the previous reactor or from the inflow sedimentgraph (for the first reactor) and C^0_{i+1} is a known value calculated during the previous time step. The average discharge Q_i is estimated by interpolating between the average discharge at the inlet and the outlet of the pond or

$$Q_i = Q_1 + (i-1)\frac{(Q_{n+1} - Q_1)}{n} \tag{57}$$

where Q_i is the average inflow discharge for the i^{th} reactor, Q_1 is the average discharge at the pond's inlet, Q_{n+1} is the average discharge at the pond's outlet, and n is the number of reactors. The average inflow discharge (Q_1) and the average outflow discharge (Q_{n+1}) are determined from the inflow hydrograph and from the outflow hydrograph developed in the program's hydraulic routing routine.

The reactor volumes in Equation 56 (V^0_{i+1} and V^n_{i+1}) are defined as

$$V^{(t)}_{i+1} = (PV(t) - DEAD)/n \tag{58}$$

where PV(t) is the pond volume at time t, DEAD is the volume of permanent pool that is dead space, and n is the number of reactors. PV(t) is determined and stored by the hydrograph routing procedure. Equation 58 is used to divide the pond volume equally among each reactor.

To understand the deposition term (DEP) of Equation 56, it is useful to discuss the differences between settling of particles in a plug flow model such as DEPOSITS and that of a mixing model such as

CSTRS. In a plug flow model, each particle of a given inflow slug of sediment (a slug of sediment is defined as the amount of sediment to enter the pond within the time increment used in Equation 56) has the same detention time and thus a single time parameter can be used to characterize the settling time. For a CSTRS model, inflow sediment mixes with sediment from previous inflow slugs; therefore, a given sediment discharge from the pond will contain some mass from the current inflow slug as well as some from previous inflow slugs. Consequently, a single time parameter cannot be used to represent the residence time of sediment particles contained in each inflow slug.

To handle a variation in the residence time of individual particles in the CSTRS model, a mass balance is maintained for each inflow slug of sediment. This mass balance can be written as

$$M_{Rj}^{i} = M_{Rj}^{i-1} - SDEP_{j}^{i} - DM_{j}^{i} \tag{59}$$

where M_{Rj}^{i} is the mass of the j^{th} sediment slug at the end of the i^{th} time step; M_{Rj}^{i-1} is the mass of the j^{th} sediment slug at the end of the previous time step; $SDEP_j^i$ is the amount of sediment from the j^{th} slug that is deposited within the i^{th} time step; and DM_j^i is the amount of sediment from the j^{th} slug that is discharged from the pond in the i^{th} time step. The maintenance of this mass balance indirectly determines the residence time of individual particles, and hence is useful in calculating sediment deposition. Although the final result of this algorithm is relatively straightforward, the mathematical development of SDEP and DM is quite tedious. Details of this development are given in Wilson, Barfield, and Warner (1983).

The accuracy of the CSTRS model was evaluated by using data collected by Tapp, Barfield, and Griffin (1981) in a pilot scale apparatus. The purpose of the Tapp study was to evaluate the benefits of flocculants in enhancing the trap efficiency of sediment ponds.

However, in the course of these studies, six data sets were collected in which flocculants were not used. These six runs were conducted with variable inflows at the peak inflow rates and peak influent concentrations noted previously in Table 14. The CSTRS model used to generate the predicted trap efficiencies shown in the table had two continuous stirred reactors and an assumed dead space volume of 15%. As can be seen in Table 14, this study showed that the DEPOSITS and CSTRS models predicted trap efficiency with approximately the same degree of accuracy.

Predicted and observed effluent concentrations for the Tapp, Barfield, and Griffin data are shown in Figures 32 through 37. The figures show that the CSTRS model does a good job of predicting the shape of the observed sedimentgraph and does a fair job of predicting the peak effluent concentrations. The DEPOSITS model does, at best, a fair job of predicting the shape of the observed sedimentgraph, but is roughly equivalent to the CSTRS model in predicting the peak effluent concentration. Overall it appears that the CSTRS model is better than the DEPOSITS model for predicting sedimentgraph values.

2.6.2.3.8 The BASIN reactor diffusion model. One of the deficiencies of the DEPOSITS and CSTRS models is the assumption of no sediment resuspension. In order to include an estimate of resuspension in a model, it is necessary to have an algorithm to calculate diffusion in a pond. An algorithm was developed by Wilson, Barfield, and Colliver (1984) to model sediment movement in detention ponds considering sediment deposition, sediment resuspension due to turbulence, bed degradation, and sediment dilution by mixing of sediment-laden inflow with previously stored flows. The first three components are based on established physically based concepts. The mixing between sediment-laden inflow and previous flows, however, is determined conceptually. The complete pond algorithm is called BASIN, which is an acronym for basin analysis of sediment-laden inflow.

In BASIN, the pond is divided into a series of chambers of equal volume as shown in Figure 38 analogous to a series of reactors

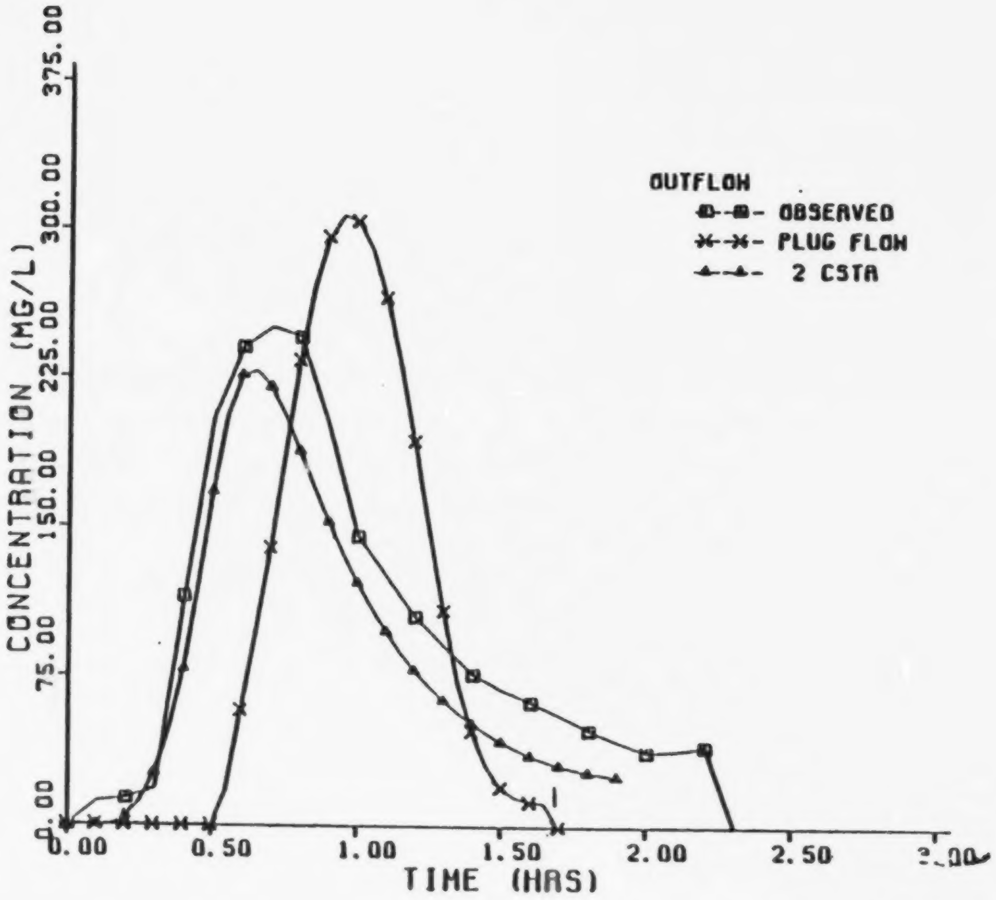


Figure 32. Pond run 18. (Wilson, Barfield, and Moore 1982)

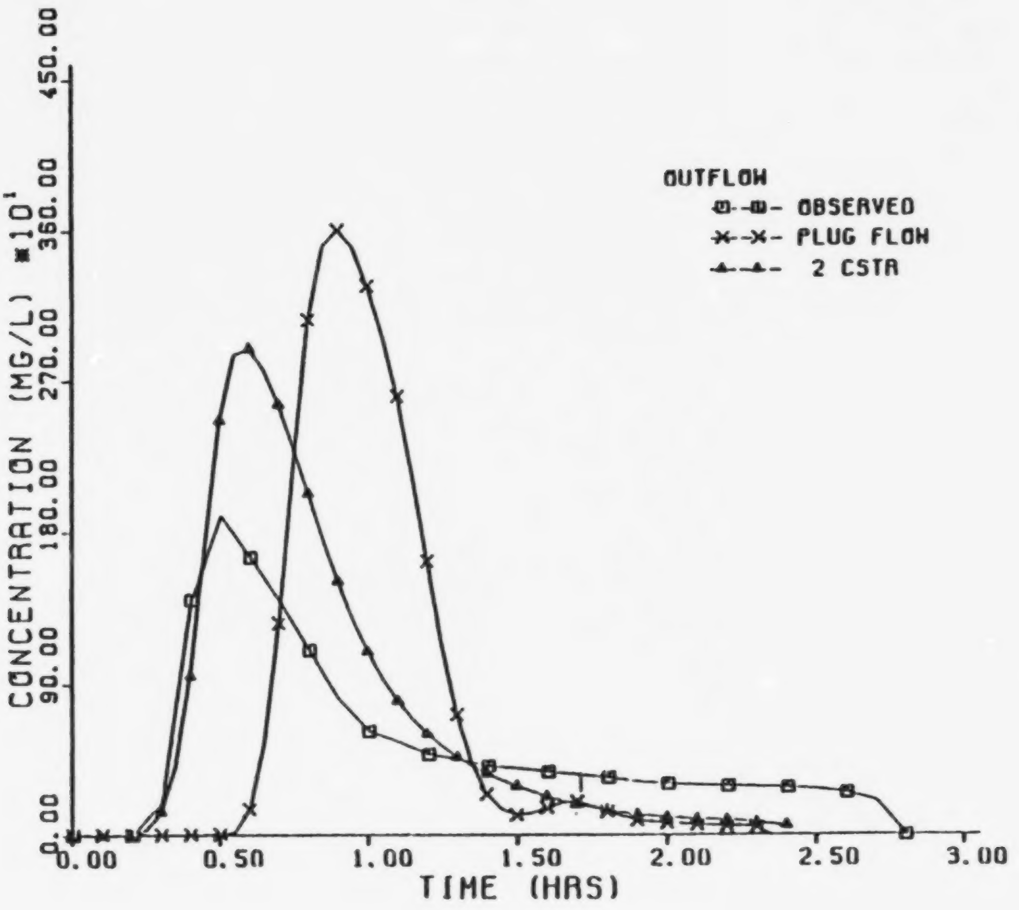


Figure 33. Pond run 19. (Wilson, Barfield, and Moore 1982)

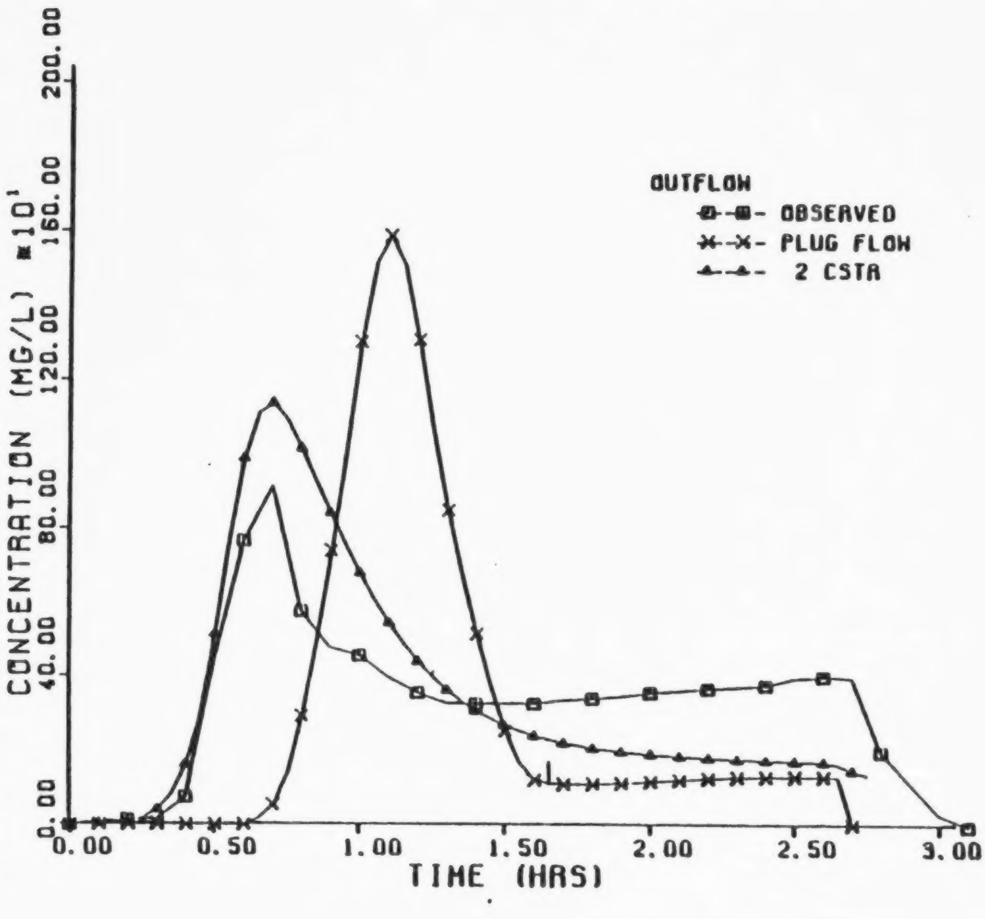


Figure 34. Pond run 22. (Wilson, Barfield, and Moore 1982)

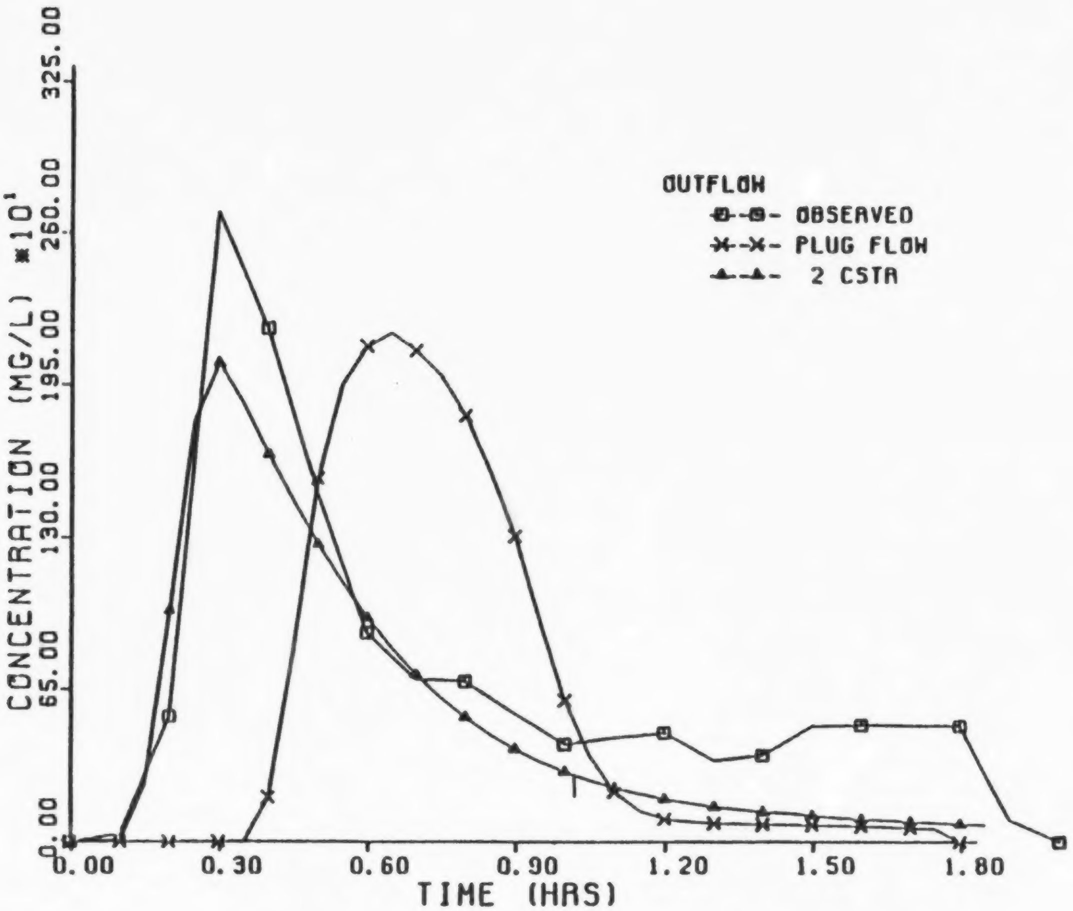


Figure 35. Pond run 24. (Wilson, Barfield, and Moore 1982)

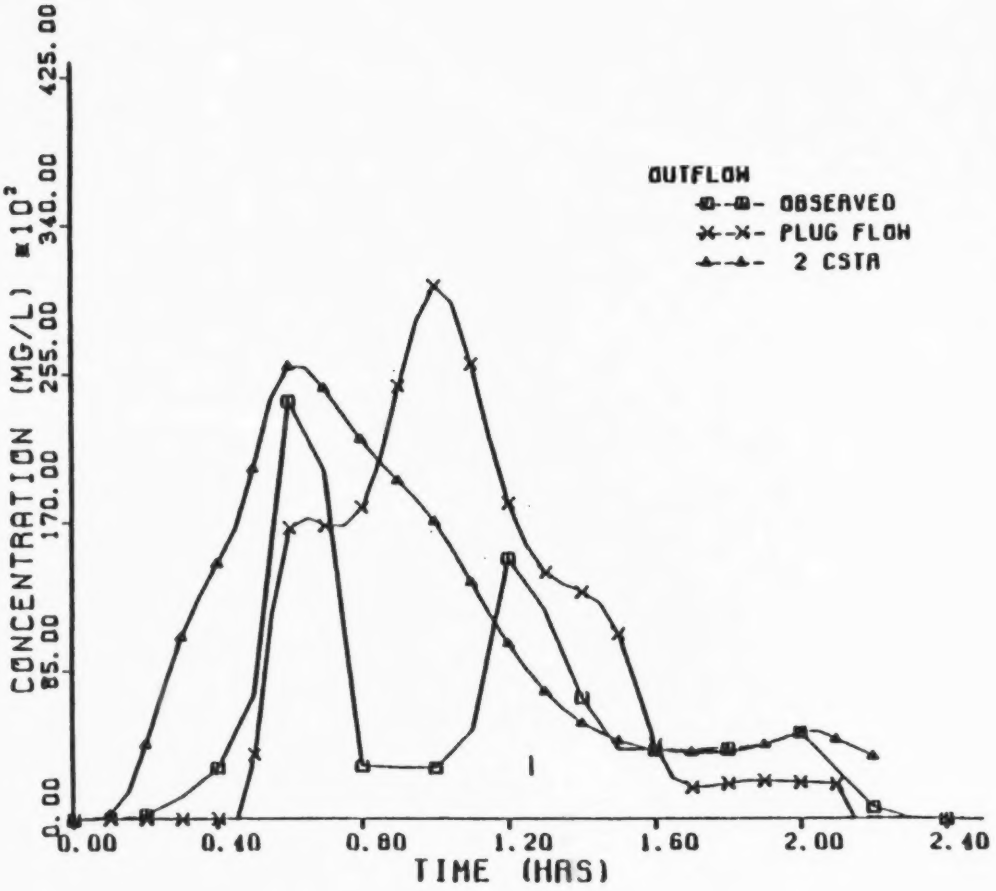


Figure 36. Pond run 26. (Wilson, Barfield, and Moore 1982)

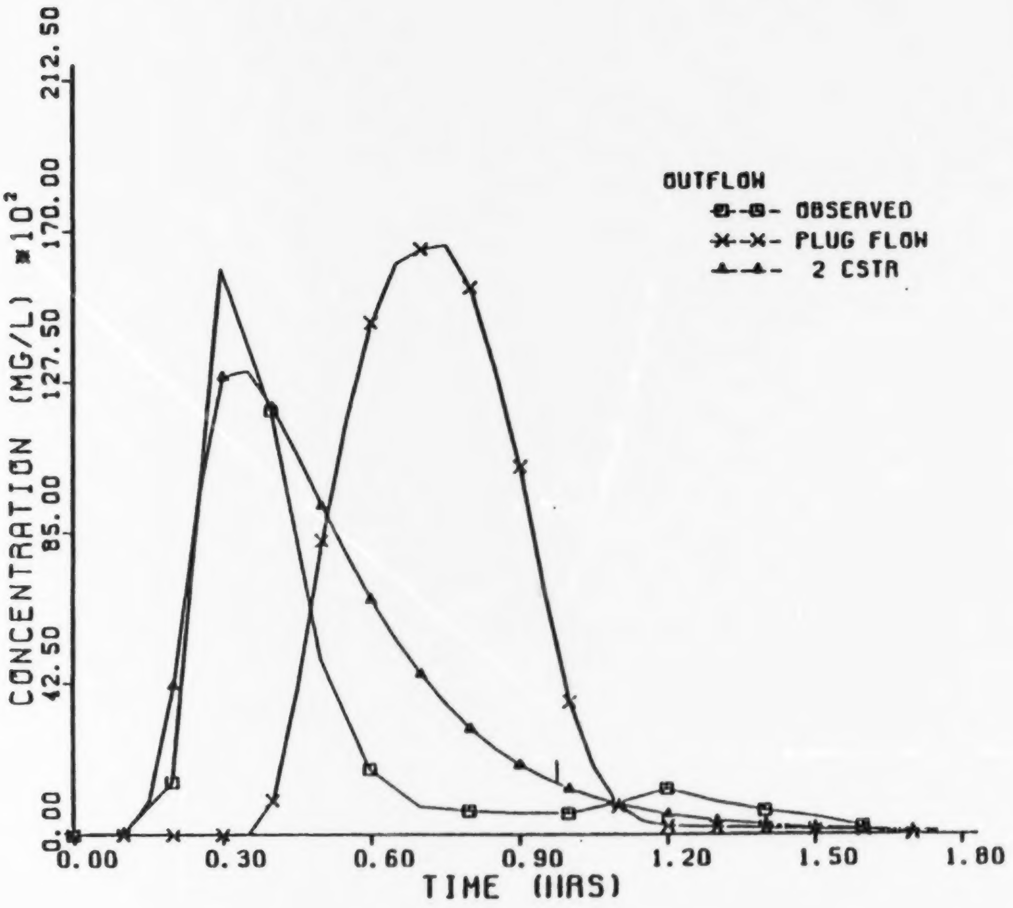


Figure 37. Pond run 28. (Wilson, Barfield, and Moore 1982)

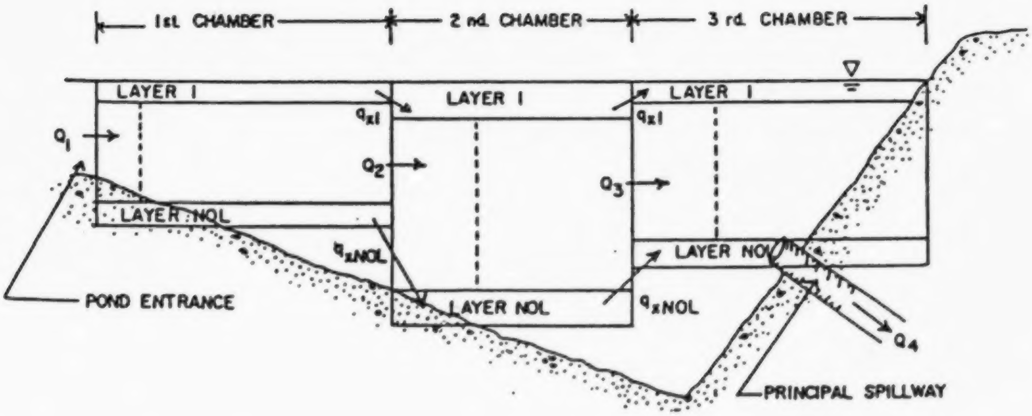

1st CHAMBER
2 nd CHAMBER
3 rd CHAMBER
LAYER I
LAYER I
LAYER I
Q1
qx1
Q2
qx1
Q3
LAYER NOL
qxNOL
LAYER NOL
LAYER NOL
qxNOL
Q4
POND ENTRANCE
PRINCIPAL SPILLWAY

Figure 38. Schematic representation of a pond as used in the BASIN model. (Wilson, Barfield, and Colliver 1984)

in CSTRS. A sediment mass balance is then conducted on a layer within a given chamber. This expression is greatly simplified by using the characteristics of reactor theory and by idealizing the hydraulic flow pattern. Bed scour rate is predicted by modifying the approach of Einstein (1950). Diffusion between layers and from the bed is calculated by a diffusivity gradient equation.

By using reactor theory to predict horizontal mixing only, the problem of predicting circulation patterns in a pond is eliminated while still allowing for diffusion in the vertical direction. Thermal stratification, however, is not considered.

Wilson, Barfield, and Colliver (1984) collected data in a pilot scale apparatus to test the BASIN model. The results are summarized in Table 16 and details are available in Wilson, Barfield, and Colliver (1984) and Wilson (1984). In general, the results show that BASIN overpredicts effluent concentrations at high TSS values. Unpublished results with the CSTRS model showed the same trend. At lower inflow concentrations (less than 25 000 mg/L), results were quite good. The probable cause of the prediction error is the sweeping action that occurs at high inflow concentrations when sand sized particles sweep out smaller colloidal particles.

At the present time, the BASIN model is not included in an operational watershed hydrology model. Details on required inputs are given in Section 2.6.2.6.

2.6.2.4 Physically based models: hydrodynamic models. Hydrodynamic models are three-dimensional flow models that solve the equations of motion in a pond by considering the conservation of mass and momentum. When coupled with a sediment continuity equation, the resulting series of partial differential equations can theoretically be solved numerically for effluent sediment concentrations, flow conditions, and boundary conditions. Unfortunately, the computational times required and the complexity of the inputs usually preclude the use of these models on a practical basis. Therefore, the following discussion provides only a brief outline of some of the basic principles associated with hydrodynamic models.

Table 16. Summary of predicted and observed results for tests of the BASIN model.

Run No.	Peak Inflow Concentration (mg/L)	Trap Efficiency		Peak Effluent Concentration	
		Observed (%)	Predicted (%)	(mg/L)	(mg/L)
1	86 000	70	64	27 030	31 790
2	104 000	84	79	16 050	21 800
3	91 000	84	79	16 060	20 530
4	25 000	89	87	3 320	4 480
5	19 000	75	73	4 460	4 490
6	15 000	79	82	2 810	2 640

Three-dimensional flow models are developed by applying Newton's second law of motion with the conservation of mass within the flow domain. The conservation of mass for an incompressible fluid can be written as

$$\frac{u_i}{x_i} = 0 \qquad (60)$$

and Newton's second law of motion can be written as

$$\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = \frac{-1}{p} \frac{\partial p}{\partial x_i} + \frac{\mu}{p} \frac{\partial^2 u_i}{\partial x_j \partial x_j} + g_i \qquad (61)$$

where u_i are the x_i components of the velocity vectors, g_i is the gravity force term, p is the fluid density, p_r is the fluid pressure, m is the dynamic viscosity, and t is time. Equations 60 and 61 are written using tensor notation (Currie 1974). Details of the derivations of equations 60 and 61 can be found in any intermediate fluid mechanics textbook (e.g., Currie 1974; Schlichting 1979; Bird, Stewart, and Lightfoot 1966). Equation 61 is know as the Navier-Stokes equation, which, when combined with the conservation of mass, provides four equations to solve for the four unknowns: u_1, u_2, u_3, and p_r.

Theoretically, the Navier-Stokes equations can be used to solve any fluid flow problem involving Newtonian fluids. For laminar flow, analytical and numerical solutions to these equations have been successfully applied to a variety of flow conditions (Schlichting 1979). For turbulent flows however, the Navier-Stokes equations remain essentially unsolvable. Analytical solutions are rare because of the three-dimensional nature of turbulence. Thus, convenient simplifications into one- or two-dimensional flows are not possible.

Turbulent flows are characterized by the pseudo-random motion of large masses of fluid typically known as eddies. As the scale of motion is reduced, the flow eventually becomes laminar. Numerical solutions at this level are impractical because of the small time and space scales associated with such flows (Clark, Ferziger, and Reynolds 1979; Gibson et al. 1981). Therefore, to describe turbulent flow problems, the Navier-Stokes equations are averaged with respect to time or, more recently, with respect to space.

Time-averaged turbulent flow equations are obtained by dividing the velocity and pressure terms into a mean component and a fluctuating component (e.g., $u_i = U_i + u'_i$). These components are then substituted into equations 60 and 61 and averaged with respect to time. The resulting time-averaged equations are determined as (see Tennekes and Lumley 1972; Schlichting 1979)

$$\frac{\partial U_i}{\partial x_i} = 0 \tag{62}$$

and

$$\frac{\partial U_i}{\partial t} + U_j \frac{\partial U_i}{\partial x_j} = \frac{-1}{p} \frac{\partial P}{\partial x_i} + F_{bi} + \frac{\mu}{p} \frac{\partial^2 U_i}{\partial x_j \partial x_j} - \frac{1}{p} \frac{\partial p u'_i u'_j}{\partial x_j} \tag{63}$$

where U_i is the mean velocity component, u'_i is the fluctuating velocity component, P is the mean pressure component and F_{bi} is the mean gravitational force. The statistical correlation terms $pu_i'u_j'$ are known as Reynolds' stresses and are formed by the non-linear velocity terms in Equation 63. The Reynolds' stresses introduce an additional nine unknown terms. The resulting problem is known as a closure problem.

A large assortment of models has been proposed to close the above system of time-averaged equations, some of which are very complex. None of these models are completely satisfactory since they depend on empirically fitted coefficients. Turbulent modelling techniques can be grouped into Reynolds' stress transport models, two-equation models, one-equation models, and zero-equation models (Launder and Spalding 1972; Reynolds 1976; Gibson et al. 1981). The latter group of models is classified by the number of additional partial differential equations. A review of these models is given by Wilson (1984).

2.6.2.5 Physically based models: other models

2.6.2.5.1 The USDA FARMPND model. As a component of the new USDA Small Watershed Model (SWAM), an algorithm is being developed to predict sediment trapping and chemical removal processes in small lakes (Gallegos, Schiebe, and Stefan 1983; Schiebe 1983). The model is scheduled to be released in late 1985 or early 1986 and although complete details are not available at the present time, the objectives of the FARMPND modelling effort are fairly complete. The model is being developed to respond to the following variables:

1. Inflow rates,
2. Sediment concentrations,
3. Pond geometries,
4. Sediment size distributions, and
5. Temperature profiles.

In addition to sediment predictions, the model will also consider other conservative material balances as well as non-conservative material balances (i.e., phosphorous, nitrogen, etc.).

According to Schiebe (1983), sediment trapping will be calculated by using a size distribution and a function describing the probability of a particle being trapped. Apparently, the probability function is to be developed from calculations with theoretically based analytical functions such as those of Dobbins (1944) and Camp (1944).

No details on the development procedure are available at the present time.

The temperature predictions will follow procedures developed by Stefan, Dhanothanan, and Schiebe (1984) and Ford and Stefan (1980). The model will be a continuous simulation model as opposed to a single event model.

2.6.2.5.2 The CREAMS impoundment model. The USDA CREAMS model (Knisel 1980) was developed to predict non-point source pollution from agricultural sized fields as affected by best management practices. The impoundment element is that proposed by Laflen et al. (1978) for modelling the sediment trapping that occurs in level terraces with tiled outlets. The impoundment would typically be on the order of 0.2 hectares or less.

In the impoundment model, runoff is assumed to instantly fill the basin in a completely mixed state. Sedimentation is assumed to occur by discrete quiescent settling and water is assumed to discharge from the bottom of the impoundment. Particles are lost from the impoundment by either being discharged through the outlet, settling to the bottom, or infiltrating into the pond bottom. The effects of thermal stratification are not considered.

2.6.2.6 Summary tables for pond model inputs. Tables are given in this section that summarize some of the basic features and requirements of the more popular design models presented in the preceding discussion. The table for each model reviews basic operating principles, predicted outputs, input requirements, probable sources of input data, and qualitatively describes the expected accuracy of the model's sediment removal efficiency estimates. In addition, each table provides a listing of the large-scale hydrology - sedimentology models that are known to use the given pond modelling routine. These large-scale hydrology-sedimentology models are discussed in more detail in Section 2.7.

Summary tables 17 to 25 are presented in order of sophistication with the simplest models appearing first and the most elaborate models, requiring the most input parameters, appearing last. For a given quality of input data, the accuracy of sediment removal efficiency estimates can be expected to improve as the sophistication of the model selected increases. It should be noted, however, that as the level of model sophistication increases, so do the difficulties associated with model implementation and input data definition.

2.7 COMPLETE WATERSHED MODELS

Complete watershed models are design tools that are used not only to evaluate the performance of settling ponds but also to characterize the flow and sediment yield from the contributing watershed. They are most commonly used in the form of computer programs that have various algorithms to predict the effects of variations in watershed conditions on runoff. The most effective watershed models account for all of the following processes or inputs:

1. Input rainfall distribution,
2. Rainfall abstractions (i.e., surface storage, infiltration),
3. Overland flow and watershed storage,
4. Channel routing and storage,
5. Reservoir routing and storage,
6. Soil erosion,
7. Sediment yield,
8. Channel routing and erosion, and
9. Reservoir deposition.

The hydrologic portion of the models serves [illegible] driving force for the sedimentology routines. For example, ch[illegible] depends heavily on the runoff rate. Thus, the sediment[illegible] not stand alone and predictions from the sedimentology routine are [illegible] accurate as the predicted hydrologic values. Two models that use the same sedimentology routines, but different hydrologic routines, will give different sediment yield values for the same watershed

Table 17. Summary of the Brune sedimentation model.

Model Operating Principles

The model is an empirical curve relating trapping efficiency to the ratio of storage capacity to annual inflow (C/I ratio). The C/I ratio can be roughly interpreted as detention time. The predictions do not consider particle sizes. Experimental results indicate that the model is a poor predictor of trapping efficiency for small sediment basins.

Predicted Output

Trapping efficiency

Watershed Models Incorporating Procedure

HEC 6

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1. Impoundment capacity	1. User input
2. Average annual inflow	2. User input

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 18. Summary of the Churchill sedimentation model.

Model Operating Principles

The model is an empirical curve relating trapping efficiency to the ratio of retention time to mean velocity. The model does not consider particle size effects. No data are available testing the model on sediment ponds.

Predicted Output

Trapping efficiency

Watershed Models Incorporating Procedure

Unknown

Input Requirements

Input Parameters	Probable Sources of Input Parameters
1. Retention time	1. Hydraulic routing model
2. Mean velocity	2. Hydraulic routing model

Table 19. Summary of steady state, quiescent flow, overflow rate sedimentation models (including USEPA model).

Model Operating Principles

Trapping efficiency is predicted by the simple overflow rate model with a single overflow rate used for the entire storm. Tapp, Barfield, and Griffin (1981) found that the predictions of effluent TSS concentrations were erratic.

Predicted Output

Trapping efficiency
Average effluent TSS concentration
Average effluent size distribution (can be determined from data)

Watershed Models Incorporating Procedure

SWMM

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1. Peak discharge or average discharge	1. Hydraulic routing model
2. Inflow particle size distribution	2. Watershed sedimentology model or user input

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 20. Summary of unsteady, quiescent flow, overflow rate sedimentation models (including Tapp's methods).

Model Operating Principles

Trapping efficiency is calculated by the overflow rate method for each plug of flow in a reservoir. The overflow rate is variable for each plug. Tapp, Barfield, and Griffin (1981) found that predictions of effluent concentration were not accurate, but trapping efficiency predictions were adequate.

Predicted Outputs

Trapping efficiency for each plug
Effluent TSS concentration for each plug
Effluent particle size distribution
Overall trapping efficiency

Watershed Models Incorporating Procedure

Curtis and McCuen Model
SWMM

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1. Detention time by plugs	1. Reservoir hydraulic routing
2. Inflow sedimentgraph	2. Watershed sedimentology model or user input
3. Inflow particle size distribution	3. Water sedimentology model or user input

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 21. Summary of unsteady, turbulent flow, overflow rate sedimentation models.

Model Operating Principles

Same as Table 20 except turbulence is considered by using the Dobbins-Camp curves or Chen's curves. Tapp, Barfield, and Griffin (1981) found that in many cases the use of turbulent parameters did not improve the accuracy of sediment removal efficiency predictions.

Predicted Outputs

Same as Table 20

Watershed Models Incorporating Procedure

SWMM

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1-3 Same as Table 20	1-3 Same as Table 20
4. Turbulent Peclet No.	4. Input parameter

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 22. Summary of the DEPOSITS sedimentation model.

Model Operating Principles

Both a hydraulic and sediment routing model. Inflow and outflow hydrographs are divided into plugs and the detention time and average depth for each plug determined. Each plug is divided into layers and discrete settling theory is used to estimate sediment removed from each layer based on the size distribution during the detention time of each plug. Based on the withdrawal distribution, an effluent concentration and size distribution is calculated. Predictions of trapping efficiency have been accurate, but predictions of the timing and magnitude of effluent concentrations have been erratic in tests to date.

Predicted Outputs

Effluent TSS concentration for each plug
Effluent particle size distribution
Effluent settleable solids (for some versions of the model)
Settling pond sediment trapping efficiency

Watershed Models Incorporating Procedure

SEDIMOT II
Smith Model (1981)
USGS Model

continued...

Table 22. Concluded.

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1. Basin Geometry	1. User input
2. Stage discharge curve	2. User input
3. Inflow hydrograph	3. Watershed hydrology model or user input
4. Inflow sedimentgraph	4. Watershed sedimentology model or user input
5. Inflow particle size distribution	5. Watershed sedimentology model or user input
6. Non ideal settling parameters (short-circuiting, turbulence, dead storage)	6. User inputs

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 23. Summary of the CSTRS sedimentation model.

Model Operating Principles

Both a hydraulic and sediment routing model. The basin is divided into a series of 'n' continuous stirred reactors. Using a modification of CSTRS reactor theory, the effluent concentration is estimated for the 'nth' reactor and used as the discharge from the basin. Evaluations on pilot scale ponds show that the model accurately predicts effluent concentrations and trapping efficiencies for normal inflow concentrations, but overpredicts concentrations for high inflow concentrations (100 000+ mg/L).

Predicted Outputs

Same as Table 22

Watershed Models Incorporating Procedure

SEDIMOT II

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1-6 Same as Table 22	1-6 Same as Table 22
7. Number of CSTRS	7. User input. Recommended value is 2.0

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 24. Summary of the BASIN sedimentation model.

Model Operating Principles

Same as CSTRS except each reactor is divided horizontally into 10 layers and diffusion calculated in the vertical dimension. Resuspension is estimated by Einstein's method. The model predictions are similar to those of CSTRS, but in the case of shallow flows with high velocities, resuspension is predicted.

Predicted Outputs

Same as Table 22

Watershed Models Incorporating Procedure

Will probably be included in SEDIMOT II at a future date

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1-7 Same as Table 23	1-7 Same as Table 23
8. Turbulent diffusivity	8. User input

---

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Table 25. Summary of the CREAMS sedimentation model.

Model Operating Principles

Model developed for small tile drained impoundments on parallel terraces where all flow is assumed to enter at once. Settling calculated similar to DEPOSITS. Field tests of prediction accuracy limited to comparison of predicted and observed total sediment mass discharged in a storm. Over 16 storm events, the ratio of standard error to average discharge was over 100% (SE of 0.323 tons with an average sediment discharged of 0.269 tons).

Predicted Outputs

Discharge pollutograph
Discharge size distribution

Watershed Models Incorporating Procedure

CREAMS

continued...

Table 25. Concluded.

Input Requirements

Input Parameters[a]	Probable Sources of Input Parameters
1. Drainage area above ponds	1. User input
2. Infiltration rate in pond	2. User input
3. Impoundment geometry and outlet characteristics	3. User input
4. Storm discharge into pond	4. Watershed hydrology model
5. Sediment yield into pond	5. Watershed sedimentology model
6. Particle size distribution of inflow sediment	6. Watershed sedimentology model

[a] When used as a subroutine in a watershed model, the user has to develop the required inputs for the watershed model.

Tables 26 to 30 summarize some of the more commonly referenced watershed models. This list is not intended to be exhaustive, but is presented to illustrate the types of models available and some of their capabilities. Particular emphasis is placed on those models with potential application to disturbed area sediment pond design. Thus only those models that have a hydrology, sedimentology, and sediment pond routine are described. Several other well- known hydrology-sedimentology models were not included in this summary due to the lack of an impoundment element. Among these excluded models are the Purdue ANSWERS model (Beasley and Huggins 1982), the VPI FESHM model (Ross, Shanholtz, and Contractor 1980), the USDA SPNM model (Williams 1981), and the Corps of Engineers STORM model (U.S. Army Corps of Engineers, Hydrologic Engineering Center 1976).

2.8 COST CONSIDERATIONS

2.8.1 Introduction

This section examines the variation in pond capital costs that can result from the use of different sedimentation design models. This evaluation was completed by designing a settling pond for a fictitious watershed using four of the sedimentation models presented in Section 2.6. The costs of the four pond sizes predicted were then estimated and presented to show how variations in design model sophistication affect final pond capital costs.

The four sedimentation models used in the cost evaluation were:

1. Brune's method,
2. USEPA overflow rate model,
3. DEPOSITS reactor model, and
4. CSTRS reactor model.

The last two reactor models were used within the framework of the SEDIMOT II watershed model.

Table 26. Description of the SEDIMOT II watershed model.

Model Type and Intended Use

SEDIMOT is an acronym for sedimentology by distributed modeling techniques. It is a single storm model intended for use in designing sediment control structures for surface mined watersheds and analyzing the hydrologic consequences of mining.

Hydrology Routines

Rainfall Distribution:

1. SCS Type I,
2. SCS Type II, or
3. Input rainfall distribution.

Rainfall Abstractions:

SCS curve No. accounts for all abstractions.

Overland Flow Routing Techniques:

Unit hydrograph based on time of concentration for timing parameters. Unit hydrograph shape changes based on land use.

Channel Routing Techniques:

Muskingham's Method.

Reservoir Routing Techniques:

Puls continuity routing. The model calculates stage discharge curve for drop inlet spillways.

Sedimentology Predictions Methods

Soil Erosion and Sediment Yield Techniques:

1. Williams MUSLE (Williams 1975), or
2. SLOSS, which uses the CREAMS erosion equations (Knisel 1980).

Sediment Routing in Channels:

1. Williams routing equations (William 1975),or
2. Routing based on stream power (Yang and Stall 1976).

continued...

Table 26. Concluded.

Reservoir Sedimentation Routine:

1. DEPOSITS model (see Table 22), or
2. CSTRS model (see Table 23).

Other Sediment Control Routines:

1. Grass filters, and
2. Porous check dam.

Comments

1. The model has a user's guide summarizing input information.
2. The model has been adopted for use in many American states.

References

1. Wilson, Barfield, and Moore (1982)
2. Wilson, Barfield, and Colliver (1984)
3. Wilson et al.(1984)

Table 27. Description of the CREAMS nonpoint source model.

Model Type and Intended Use

CREAMS is an acronym for chemicals, runoff, and erosion from agricultural management systems. It is a model intended for field sized areas to evaluate nonpoint pollution as affected by best management practices. The model is a daily continuous simulation model but can consider breakpoint rainfall.

Hydrology Routines

Rainfall Distribution:

1. None specified for daily rainfall.
2. Input breakpoint rainfall distribution.

Rainfall Abstractions:

1. SCS curve No. for daily rainfall, or
2. Green Ampt equation (Green Ampt 1911) for breakpoint rainfall.

Overland Flow Routing Techniques:

1. None used for daily rainfall; peak discharge estimated by a regression equation.
2. Kinematic wave theory for breakpoint rainfall.

Channel Routing Techniques:

Regression equations based on the spatially varied flow equation.

Reservoir Routing Techniques:

Apparently a continuity routing. The manual is not clear.

Sedimentology Prediction Methods

Soil Erosion and Sediment Yield Techniques:

1. Overland flow detachment; Foster, Meyer, and Onstad (1977) model which separately calculates rill and interill erosion.
2. Channelized flow erosion; Foster (1982) equations for channel erosion based on shear distribution.

continued...

Table 27. Concluded.

Sediment Routing in Channels:

Comparison of sediment available for transport with the Yalin sediment transport equation (Foster and Meyer 1972).

Reservoir Sedimentation Routine:

CREAMS impoundment element (see Table 25).

Other Sediment Control Routines:

Impoundment terraces.

Comments

1. The model has a user's guide summarizing user inputs.
2. The model is used in the United States for nonpoint pollution evaluation.

References

Knisel (1980)

Table 28. Description of the USEPA stormwater management (SWMM) watershed model.

Model Type and Intended Use

SWMM is a model developed to simulate the quality and quantity of stormwater flow from urbanized areas. The model can be a single storm model or continuous rainfall simulation model.

Hydrology Routines

Rainfall Distribution:

1. Daily rainfall, and
2. User-specified breakpoint.

Rainfall Abstractions:

1. Infiltration
    a. Horton for daily rainfall
    b. Green Ampt for breakpoint rainfall, and
2. Surface storage as initial abstraction.

Overland Flow Routing Techniques:

1. Kinematic wave theory, or
2. Cascade of conduits.

Channel Routing Techniques:

1. Kinematic wave theory,
2. Cascade of conduits, or
3. Solution of complete equations of motion.

Reservoir Routing Techniques:

Puls continuity routing.

Sedimentology Prediction Methods

Soil Erosion and Sediment Yield Techniques:

1. Washoff routine for impervious areas, and
2. Universal Soil Loss Equation for pervious areas.

Sediment Routing in Channels:

Scour and deposition based on Shield's parameters.

continued...

Table 28. Concluded.

Reservoir Sedimentation Routine:

1. Camp-Dobbin turbulent flow equations (see Table 21), or
2. Empirical equations, user specified.

Other Sediment Control Routines:

Any user-specified treatment equation.

Comments

1. SWMM is a large FORTRAN based model that allows a great deal of flexibility.
2. The model is available free of charge and an extensive user's guide is available.
3. The model is difficult to learn to use, but a powerful tool.
4. The model does not estimate particle size distribution, but must be specified by the user.

Reference

Huber et al. (1975)

Table 29. Description of the KINEROS watershed model.

Model Type and Intended Use

KINEROS is an acronym for kinematic erosion simulation, a model developed for application to surface mine flows. KINEROS is a single storm model.

Hydrology

Rainfall Distribution:

Unspecified.

Rainfall Abstractions:

Infiltration, the only abstraction computed, is modelled by the Green Ampt (1911) equation.

Overland Flow Routing Techniques:

Kinematic Wave Theory.

Channel Routing Techniques:

Continuity routing.

Sedimentology Prediction Methods

Soil Erosion and Sediment Yield Techniques:

1. Splash detachment proportional to intensity squared;
2. Concentrataed flow detachment proportional to transport deficit; and
3. Transport capacity by one of 5 relationships.

Sediment Routing in Channels:

Unspecified.

Reservoir Sedimentation Routine:

Modified DEPOSITS program.

Other Sediment Control Routines:

None.

continued...

Table 29. Concluded.

Comments

1. Model developed for applications to surface mined lands.
2. Input values not available for concentrated flow parameters.
3. No information available in a user's guide.

Reference

Smith (1981)

Table 30. Description of the USGS DR_3M and DR_2M - QUAL watershed model.

Model Type and Intended Use

The USGS model was developed to model runoff quantity and quality in urbanized areas. It is a continuous simulation model with the capacity to respond to breakpoint rainfall.

Hydrology Routines

Rainfall Distribution:

1. Daily rainfall, or
2. User input short-term rainfall.

Rainfall Abstractions:

1. Infiltration is based on Green-Ampt (1911) equation, and
2. Surface storage, user input, must be satisfied before runoff begins.

Overland Flow Routing Techniques:

Kinematic wave routing.

Channel Routing Techniques:

Kinematic wave routing.

Reservoir Routing Techniques:

Modified Puls continuity routing.

Sedimentology Prediction Methods

Soil Erosion and Sediment Yield Techniques:

1. Impervious area - Washoff routine, and
2. Pervious areas - Williams (1975) MUSLE.

Sediment Routing in Channels:

Continuity routing, no deposition.

Reservoir Sedimentation Routine:

Combined DEPOSITS and overflow rate method.

continued...

Table 30. Concluded.

Other Sediment Control Routines:

None.

Comments

1. User's manual available with suggested user inputs.
2. Considerable flexibility available in model use.

Reference

Alley and Smith (1982)

2.8.2 Watershed Characteristics

The characteristics of the fictitious watershed used for this cost evaluation were selected to be generally representative of those in the Alberta foothills region. The disturbed site was assumed to cover an 80 ha (200 acre) area encompassing land used for a surface coal mining operation. It was assumed that the entire watershed area was either stripped, mined, or recently reclaimed. The soil in the area was presumed to be only slightly pervious and the topography to consist of slopes averaging 90 m long with a 6% gradient. It was also assumed that the only erosion control practice undertaken for the sole purpose of reducing settling pond sediment loads was furrowing of land in reclaimed areas.

The temporal precipitation distribution for a 1 in 10 year, 24-hour storm event was presumed to be adequately represented by the SCS Type II pattern. The 1 in 10 year, 24-hour precipitation was taken as 75 mm with a peak 30-minute intensity of 33 mm per hour. Average annual precipitation was assumed to be 600 mm including snowmelt.

Although the watershed characteristics selected for this exercise may not apply to all areas of the Alberta foothills the conclusions drawn with respect to relative costs are unlikely to be overly sensitive to modest variations in these characteristics.

2.8.3. Common Pond Characteristics

Each of the ponds designed for the cost evaluation used the following common dimensional parameters:

1. Minimum pond depth of 2.0 m,
2. Length to width ratio of 2 to 1, and
3. Dyke side slopes of 3 horizontal to 1 vertical.

None of the ponds included any provision for sediment storage. This was because the design models evaluated did not include algorithms for estimating sediment storage requirements. The SEDIMOT II watershed model does calculate sediment yield, but only for single storm events, and therefore cannot easily be used to predict average annual sediment yield. Sediment storage requirements would have to be estimated separately using one of the methods described in Section 2.5.1.

2.8.4 Model Input and Output Data

Tables 31 through 38 summarize input data requirements and predicted outputs for each of the design models evaluated. The rationale used for the selection of the required input parameters has also been summarized in the tables.

It should be noted that some of the input parameters for the Brune and USEPA models were derived from the DEPOSITS model output. This was done so that each design model would estimate required pond size based on the same performance specification and the same data for common input parameters. This approach was taken so that differences in model capabilities and outputs would be evaluated rather than differences in the quality of input data and the ease with which that data can be defined.

It was originally hoped that the baseline information provided by the DEPOSITS model could be based on a 50 mg/L peak effluent TSS performance specification. However, initial runs with the model showed that a peak effluent TSS of 50 mg/L was not achievable with a realistic pond size and therefore it was decided to establish a peak effluent TSS of 170 mg/L as the baseline performance specification. It should be noted that even at this elevated TSS concentration, the baseline sediment removal efficiency predicted by DEPOSITS was 99.92%.

2.8.5 Cost Estimates

Capital costs for the four settling ponds were estimated using in-house cost data relating to settling pond construction in the Alberta plains region. The cost estimates are summarized in Table 39 along with the pond surface area requirements for each of the design methods evaluated. The estimates appearing in the table were based on the following criteria:

1. All costs were calculated in first quarter 1985 Canadian dollars;

Table 31. Design model cost evaluation and summary of input data requirements for the Brune sedimentation model.

Input Parameter	Rationale for Parameter Selection
1. Average annual watershed precipitation, $P = 600$ mm (23.6")	- Given
2. Watershed CN = 85	- Assumed to be representative of disturbed cultivated land with some conservation treatment Soils presumed to be chiefly clays with high swelling potential or soils with clay pan near surface
3. Annual inflow volume = 450 000 m^3 (360 acre-ft.)	- Based on SCS runoff equation $Q = \frac{(P-0.2S)^2}{P+0.8S}$ where Q = average annual depth of water delivered from watershed in inches P = average annual depth of water applied to watershed in inches $S = \frac{1000}{CN} - 10$

continued ...

Table 31. Concluded.

Input Parameter	Rationale for Parameter Selection
4. Required trap efficiency = 99.90%	- Selected to provide the same sediment removal efficiency as the DEPOSITS model (see Table 36)

Table 32. Design model cost evaluation and summary of output data from the Brune sedimentation model.

1. Required pond capacity
   - Pond volume = 4 500 000 m^3 (3600 acre-ft.)
   - Pond area = 225 ha (555 acres)

Table 33. Design model cost evaluation and summary of input data requirements for the USEPA overflow rate model.

Input Parameter	Rationale for Parameter Selection
1. Sediment characteristics	
- Specific gravity of sediment = 2.65	- Typical value for silt/clay sediments
- Sediment particle size distribution	- Selected to be the same as that generated by SEDIMOT II for the pond influent in the DEPOSITS model.

Size (Microns)	% Finer
40	100
30	77
20	31
15	15
10	8
8	6
6	5
4	3
3	1.5
2	0.8
1.5	0.3
1	0

continued ...

Table 33. Concluded.

Input Parameter	Rationale for Parameter Selection
2. Settling pond inflow/outflow characteristics	
- Water temperature = 10°C	- Assumed minimum water temperature associated with 10 year precipitation event. (Note: This event is most likely to occur during late spring or summer months)
- Peak pond outflow = 0.15 m^3/s (5.43 cfs)	- Selected to be the same as that generated by the DEPOSITS model
3. Required trap efficiency = 99.90%	- Selected to provide the same sediment removal efficiency as the DEPOSITS model

Table 34. Design model cost evaluation and summary of output data from the USEPA overflow rate model.

1. **Required pond capacity**

   - Pond area = 2.93 ha (7.23 acres)

2. **Peak effluent TSS concentration**

   - TSS = 170 mg/L

3. **Effluent particle size distribution**

Size (Microns)	% Finer
15	100
10	100
4	60
2	15
1.5	5
1	0

Table 35. Design model cost evaluation and summary of input data requirements for the DEPOSITS reactor model.

Input Parameter	Rationale for Parameter Selection
1. Watershed characteristics	
- Area = 80 ha (200 acres)	- Given
- CN = 85	- Assumed to be representative of disturbed cultivated land with some conservation treatment - Soils presumed to be chiefly clays with high swelling potential or soils with clay pan near surface
- Time of concentration (Tc) = 1 hour	- Defined by watershed area and slope length
- Soil erodibility, K = 0.15	- Representative of well-aggregated silt-clay subsoil
- Typical slope length = 90 m	- Given
- Typical slope = 6%	- Given

continued ...

Table 35. Continued.

Input Parameter	Rationale for Parameter Selection
- CP value = 0.7	- Average value used to represent a mixture of disturbed/reclaimed land
2. Storm information	
- Duration = 24 hr	- Defined by regulatory requirements
- Return period = 10 yr	- Defined by regulatory requirements
- Precipitation depth = 75 mm	- Given
- Peak 30-minute intensity = 33 mm/hr	- Given
3. Sediment characteristics	
- Specific gravity of sediment = 2.65	- Typical value for silt/clay sediments

continued ...

Table 35. Continued.

Input Parameter	Rationale for Parameter Selection
- Sediment particle size distribution:	- Assumed overburden particle size distribution (Note: This is not the pond influent particle size distribution. The SEDIMOT II program uses this particle size distribution along with the watershed characteristics to derive the pond influent particle size distribution)

Size (Microns)	% Finer
100	100
80	95
60	89
40	70
30	50
20	20
15	10
10	5
8	4
6	3
4	2
3	1
2	0.5
1.5	0.2
1	0

continued ...

Table 35. Continued.

Input Parameter	Rationale for Parameter Selection
4. Settling pond influent characteristics	
- Water temperature = 10°C	- Assumed minimum water temperature associated with 10-year precipitation event (Note: This event is most likely to occur during late spring or summer months)
5. Settling pond characteristics	
- Pond area = 2.19 ha (5.42 acres) at high water level; pond geometry is input in the form of a stage-area curve	- Determined using an iterative process in which pond area is varied until the desired level of pond performance is achieved
- Stage area discharge curve based on drop inlet CSP principal spillway as follows:	- Determined using an iterative process in which spillway characteristics are varied until the desired level of pond performance is achieved

continued ...

Table 35. Concluded.

Input Parameter	Rationale for Parameter Selection
o surface withdrawal o 300 mm diameter riser and barrel o 15 m long o entrance loss coefficient = 1.0 o bend loss coefficient = 0.5 o weir coefficient = 0.6 o manning coefficient = 0.025 o head drop = 0.76 m	
- Dead space = 20%	- Representative value for a pond with a length to width ratio of 2:1
- 'Fix' coefficient = 1.0	- 'Fix' coefficient can be set above 1.0 to account for non-ideal settling conditions. In this case, all non-ideal conditions were assumed to be accounted for in the dead space term, therefore 'Fix' was set at 1.0

Table 36. Design model cost evaluaton; summary of output data from the DEPOSITS reactor model.

1. Storm event summary

Turbulence Factor	=	1.0	
Permanent Pool Capacity	=	14.09	Acre-Ft
Dead Storage	=	20.0	Percent
Time Increment Outflow	=	0.10	Hrs.
Viscosity	=	0.009	CM**2/SEC
Inflow Runoff Volume	=	25.2	Acre-Ft
Outflow Routed Volume	=	16.0	Acre-Ft
Storm Volume Discharged (Plug Flow)	=	4.8	Acre-Ft
Pond Volume at Peak Stage	=	33.7	Acre-Ft
Peak Stage	=	6.8	Ft
Peak Inflow Rate	=	147	CFS
Peak Discharge Rate	=	5.5	CFS
Peak Inflow Sediment Concentration	=	55024	MG/L
Peak Effluent Sediment Concentration	=	171	MG/L
Peak Effluent Settleable Concentration	=	0.0	ML/L
Peak Effluent Settleable Concentration	=	0.0	MG/L
Storm Average Effluent Concentration	=	138	MG/L
Average Effluent Sediment Concentration	=	41	MG/L
Basin Trap Efficiency	=	99.92	Percent
Detention Time of Flow with Sediment	=	36.5	Hrs

continued ...

Table 36. Continued.

Detention Time from Hydrograph Centers = 16.0 Hrs

Detention Time Including Stored Flow = 33.4 Hrs

Sediment Load Discharged = 0.89 Tons

Period of Significant Concentration = 12.0 Hrs

Volume Weighted Average Settleable Concentration during Period of Significant Concentration = 0.0 ML/L

Volume Weighted Average Settleable Concentration during Peak 24 Hour Period = 0.0 ML/L

Arithmetic Average Settleable Concentration during Period of Significant Concentration = 0.0 ML/L

Arithmetic Average Settleable Concentration during Peak 24 Hour Period = 0.0 ML/L

continued ...

Table 36. Concluded.

2. Effluent particle size distribution

Size (Microns)	% Finer
15	100
10	100
4	100
2	100
1.5	83
1	0

3. Pond outflow hydrograph

(not shown)

4. Pond outflow sedimentgraph

(not shown)

Table 37. Design model cost evaluation and summary of input data requirements for the CSTRS reactor model.

Input Parameter	Rationale for Parameter Selection
1. Watershed characteristics	
- Same as DEPOSITS model (Table 35)	-
2. Storm information	
- Same as DEPOSITS model (Table 35)	-
3. Sediment characteristics	
- Same as DEPOSITS model (Table 35)	-
4. Settling pond influent characteristics	
- Same as DEPOSITS model (Table 35)	-

continued ...

Table 37. Concluded.

Input Parameter	Rationale for Parameter Selection
5. Settling pond characteristics	
- Pond area = 2.36 ha (5.82 acres) at high water level; pond geometry is input in the form of a stage-area curve	- Determined using an iterative process in which pond area is varied until the desired level of pond performance is achieved
- Spillway characteristics	-
o same as DEPOSITS model (Table 35)	
- Dead space and 'Fix' coefficents	-
o same as DEPOSITS model (Table 35)	
- Number of reactors = 6	- Number of reactors required to provide results similar to those of DEPOSITS model

Table 38. Design model cost evaluaton; summary of output data from CSTRS reactor model.

1. Storm event summary

Turbulence Factor = 1.00
Permanent Pool Capacity = 15.32 Acre-Ft
Dead Storage = 20.0 Percent
Time Increment Outflow = 0.10 Hrs.
Viscosity = 0.009 CM**2/SEC
Inflow Runoff Volume = 25.2 Acre-Ft
Outflow Routed Volume = 15.7 Acre-Ft
Storm Volume Discharged (Plug Flow) = 15.7 Acre-Ft
Pond Volume at Peak Stage = 35.0 Acre-Ft
Peak Stage = 6.5 Ft
Peak Inflow Rate = 147 CFS
Peak Discharge Rate = 5.4 CFS
Peak Inflow Sediment Concentration = 55024 MG/L
Peak Effluent Sediment Concentration = 171 MG/L
Peak Effluent Settleable Concentration = 0.0 ML/L
Peak Effluent Settleable Concentration = 0.0 MG/L
Storm Average Effluent Concentration = 65 MG/L
Average Effluent Sediment Concentration = 65 MG/L
Basin Trap Efficiency = 99.88 Percent
Detention Time of Flow with Sediment = 16 Hrs

continued ...

Table 38. Continued.

Detention Time from Hydrograph Centers	=	16.0	Hrs
Detention Time Including Stored Flow	=	16.0	Hrs
Sediment Load Discharged	=	1.38	Tons
Period of Significant Concentration	=	38.0	Hrs
Volume Weighted Average Settleable Concentration during Period of Significant Concentration	=	0.0	ML/L
Volume Weighted Average Settleable Concentration during Peak 24 Hour Period	=	0.0	ML/L
Arithmetic Average Settleable Concentration during Period of Significant Concentration	=	0.0	ML/L
Arithmetic Average Settleable Concentration during Peak 24 Hour Period	=	0.0	ML/L

continued ...

Table 38. Concluded.

2. Effluent particle size distribution

Size (Microns)	% Finer
15	100
10	100
4	100
2	100
1.5	100
1	0

3. Pond outflow hydrograph

(not shown)

4. Pond outflow sedimentgraph

(not shown)

Table 39. Design model cost evaluation and settling pond capital cost predicted for various sedimentation models.

Design Model	Required Surface Area at High Water Level (ha)	Required Length of Embankment (m)	Required Length of Culvert (m)	Total Estimated Capital Cost (1985 $ Cdn)
Brune's Model	225.0	6355	35	2 200 000
USEPA Overflow Rate Model	2.93	736	17	500 000
Deposits Model[a]	2.19	640	17	440 000
CSTRS Model[a]	2.36	664	17	450 000

[a] Used as a subroutine in the SEDIMOT II watershed model.

2. For the USEPA, DEPOSITS, and CSTRS models the unit cost of embankment construction was taken as $675 per linear metre;
3. For the Brune model the unit cost was reduced to $340 per linear metre to account for the economies of scale realized with the relatively high volume of earthwork involved;
4. Spillway costs were based on a unit culvert charge of $200/m; and
5. All unit prices included a 35% allowance for engineering, administration, interest during construction, and contingency.

The estimates presented in Table 39 show reasonably uniform costs for the physically based models, which are all substantially lower than the cost predicted by the one empirical method (Brune's model). The unusually high cost predicted by Brune's model is indicative of the method's serious limitations. The model is simply not capable of providing reasonable estimates of the pond size required to meet a peak effluent suspended solids limitation.

The fact that the USEPA model predicts costs higher than the two reactor models (DEPOSITS and CSTRS) is consistent with the simplifications upon which the method is based. The model assumes steady flow equal to the peak 1 in 10 year, 24-hour storm flow and should therefore provide conservative estimates since the average storm flow would be considerably lower than the peak.

The difference in the costs predicted by the DEPOSITS and CSTRS models is well within the margin of variability in the figures and therefore both estimates can be considered to be essentially the same. However, the cost evaluation highlighted one inadequacy related to the CSTRS reactor model. In the original computer runs the number of CSTRS reactors was set at two to be consistent with the recommendations for new ponds discussed in Section 2.6.2.3.5. However, with this number of reactors, the predicted peak effluent TSS concentration was more than an order of magnitude higher than the 170 mg/L predicted by the DEPOSITS model. By trial and error it was found that to bring the CSTRS model TSS concentrations in line with those of DEPOSITS, the number of reactors had to be increased to six. Had the number of reactors been left at two and the pond area increased to provide a TSS equal to 170 mg/L, the resulting pond size would have been considerably greater than that predicted by the USEPA model, a finding that, as noted previously, would have been inconsistent with the assumptions upon which both methods are based. In summary, the cost evaluation calculations demonstrated that for new ponds, the difficulties associated with defining an appropriate number of reactors for the CSTRS model are substantial, and represent a significant inadequacy of the method.

2.9 CASE HISTORIES

The following sections provide examples of the use of two of the sedimentation models described in Section 2.6. The first of these case histories examines a typical application of the popular USEPA overflow rate approach, while the second presents an example of one of the more recently developed reactor models applied to a performance evaluation for an existing settling pond.

2.9.1 Overflow Rate Models

The case history for the overflow rate approach was taken from a surface coal mine, referred to herein as mine A, situated in the front ranges of west-central Alberta. This operation mines and processes metallurgical coal primarily for the export market.

The mine is located in the headwaters of a mountainous watershed drained by three principal tributary creeks which, for the purposes of this document, have been designated as creeks A, B, and C. The topographic relief over the mine lease is quite severe, with elevations ranging from 1525 m to 2600 m. Natural vegetation consists primarily of sub-alpine forest. The surficial geology consists of glacial tills and glacio-lacustrine and fluvial deposits with bedrock outcrops.

2.9.1.1 Disturbed area description. The mining operation encompasses a number of relatively deep pits. The pits intercept groundwater flow, precipitation, and runoff, which is collected in pit sumps and pumped to one of four sediment ponds currently in operation. The extent of the operation has also necessitated the diversion of the aforementioned creeks A and B. These diversions are routed through ditches and/or culverts and are retained in one of the settling ponds along with pit flows before being directed back to their natural streambed. The retention basin examined in this case history is the one servicing the creek A and B diversions and will be referred to hereafter as pond B.

Pond B, shown in Figure 39, is located in the original stream channel for creek B. The pond receives flows from the creek A and B watersheds, flows from a pit just east of the pond, and runoff from a dump pile to the south of the pond. Flow from the creek A watershed is initially retained in a natural lake (lake A) upstream of pond B. The outflow from this lake is controlled so that peak storm flows from the creek A watershed are not passed directly to pond B. Consequently, the only significant uncontrolled flow draining into pond B comes from the creek B watershed. Pond B outflow is discharged via a simple corrugated steel pipe overflow spillway to a series of channels and culverts that eventually deliver the flow to the original streambed.

2.9.1.2 Settling pond.

2.9.1.2.1 Description. Pond B was constructed in a relatively flat-bottomed portion of the original creek B streambed. The natural topography was such that a reservoir could be created by constructing a zoned earthfill dam at the base of the stream valley. The valley bottom , which is about 30 metres wide at the dam base, is bounded on both sides by steep slopes measuring 35 to 40 degrees. The dam is just over 15 metres high and was constructed with sideslopes of three horizontal to one vertical.

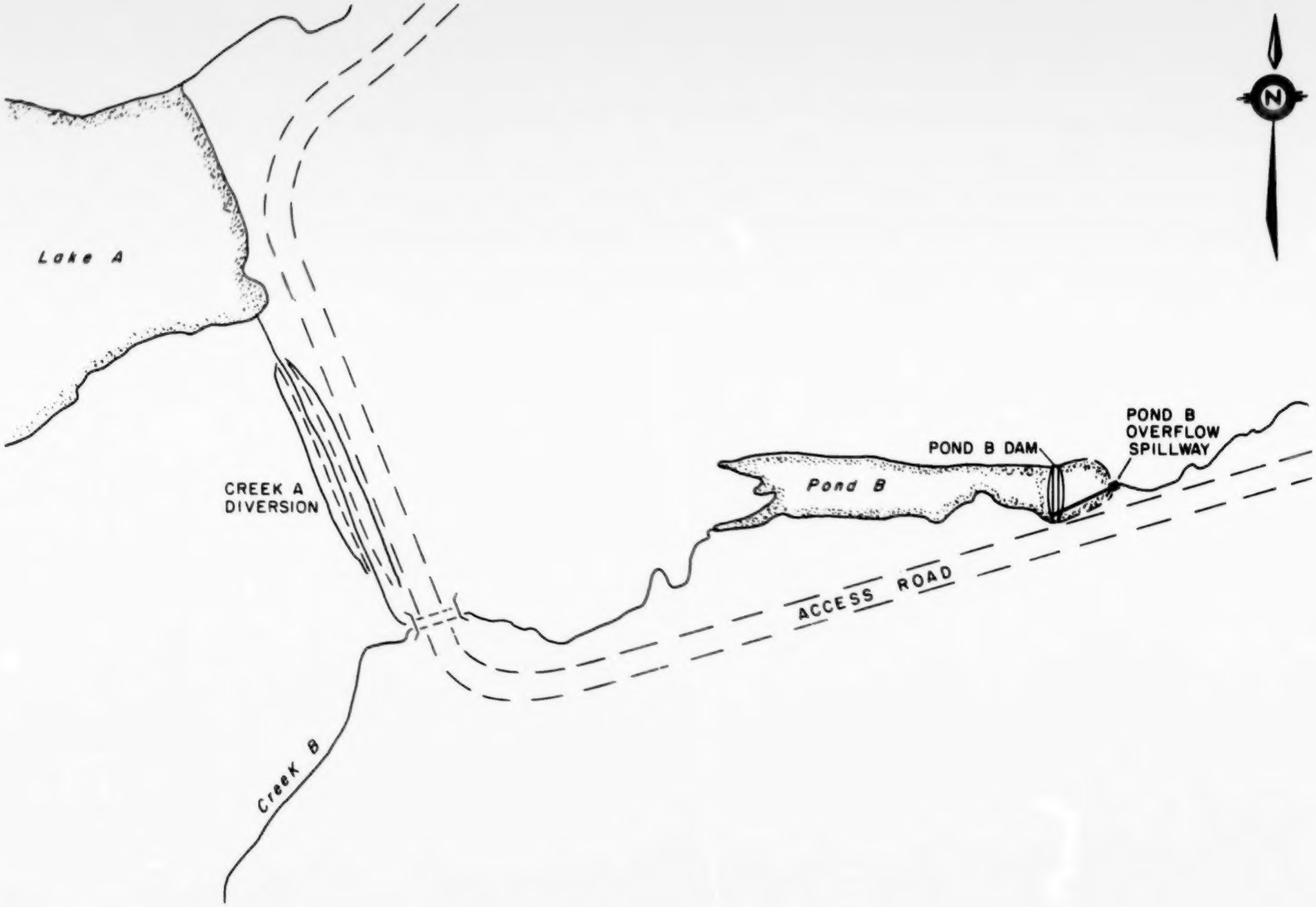

N
Lake A
CREEK A DIVERSION
Creek B
ACCESS ROAD
Pond B
POND B DAM
POND B OVERFLOW SPILLWAY

Figure 39. Case history mine A: pond B general arrangement.

The normal operating depth is greatest immediately adjacent to the dam where it is approximately 13.7 metres. At this water stage the pond has a total surface area of 1.75 ha and a length to width ratio of about 3 to 1.

The overflow spillway consists of a 1.2 metre diameter corrugated steel pipe installed immediately below the dam crest near the south abutment. Pond outflows are discharged down the dam's downstream face in a half-culvert channel with a grade of 18%. The flow's energy is then dissipated in a heavily riprapped basin before entering the diversion works.

Pond B is equipped with separate cationic and anionic polymer feed stations erected beside the pond's influent channel with the cationic station located upstream of the anionic station. Each station consists of a heated shed enclosing chemical mix and feed equipment. Dilution water used for the preparation of chemical solutions is delivered via truck to each station. Mixed polymer solutions are injected into the inlet channel via metering pumps for which rates are adjusted manually as a function of channel flows and incoming water quality.

2.9.1.2.2 Design methodology and criteria. Pond B was designed using a variation of the USEPA overflow rate approach to remove all particles larger than 20 microns in diameter at a flow equal to the peak pond inflow resulting from a 1 in 10 year, 24-hour storm. This peak discharge was established by considering the effects of both controlled and uncontrolled pond inflows. Controlled pit flows were based on the maximum pumping capacity of approximately 0.23 m^3/s while the uncontrolled flows from the creek B watershed were derived from watershed area-discharge rating curves developed during a previous hydrologic study. Storm flows from the creek A watershed were assumed to be stored in lake A and therefore were not included in the pond flow determinations.

The required pond surface area was calculated based on the 20 micron removal criteria and a non-ideal settling factor of 1.3

rather than the typical value of 1.2 noted in the discussion presented in Section 2.6.2.2.2. It was assumed that particles smaller than 20 microns in diameter would be removed by the injection of polymer flocculants. Dosages were to be set at the level necessary to provide effluent satisfying the requirements of the 'Alberta Coal Mining Waste Water Effluent Guidelines' (see Table 1).

2.9.1.2.3 Operational history. Pond B has been in operation since late 1982 and is currently slated for decommissioning in the middle of 1985. The pond is presently about two-thirds full of sediment, however the decommissioning date was established by the life of the mine pit pond B services, not the expected date for exhaustion of its sediment storage capacity.

The pond operators have noted that as inflows increase, influent TSS concentrations go up and the incoming sediment particle size distribution tends to shift toward finer gradations. They have also noted that the majority of fines appear to originate from the controlled pit flows. Figure 40 shows the variation in sediment particle size distribution that has been measured for the combined influent to pond B. The large component of minus 20 micron sediment shown in the figure is typical of the fine material pumped from the pit, which contributes approximately 90 to 95% of pond B's total sediment load. Influent suspended solids concentrations have been observed to range anywhere from 100 mg/L to over 9000 mg/L during storm events.

The pond, operating in conjunction with its chemical feed system, is generally very successful at limiting effluent TSS concentrations to 50 mg/L. The combined facility typically achieves sediment removal efficiencies in the order of 97% to 100% and has compliant effluent approximately 85% of the time. The polymer dosages required to maintain these efficiencies normally range from 10 mg/L to 20 mg/L for cationic polymer when combined with 1 mg/L to 3 mg/L of anionic polymer.

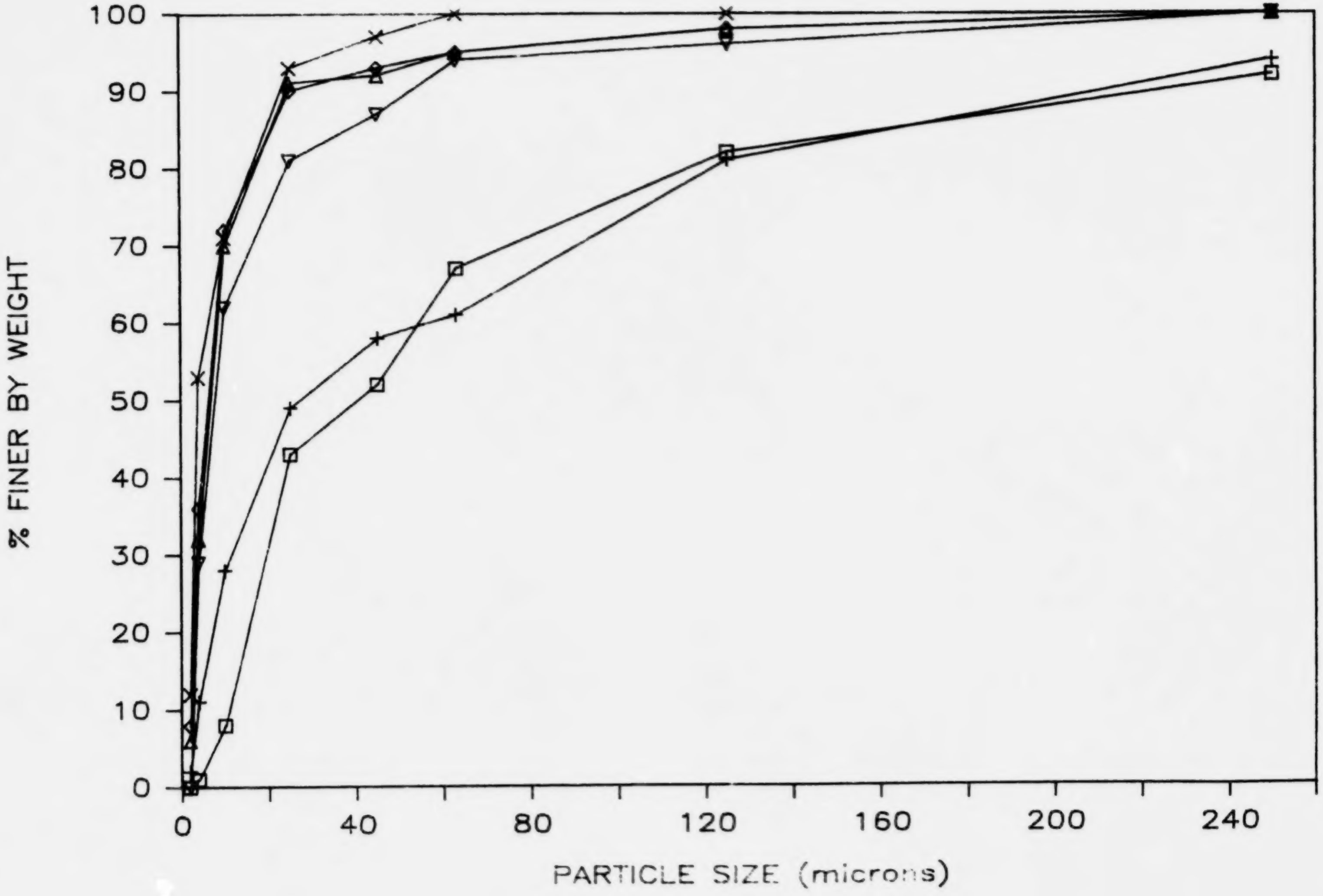


Figure 40. Case history mine A: natural watershed particle size distributions

During the winter, ice covers of up to one metre have been observed on the pond's surface. This depth of ice does not appear to impair effluent quality or reduce sediment removal efficiencies as long as 'under ice' flow is maintained. Should the ice freeze over completely, flows are short-circuited over the ice resulting in markedly increased effluent TSS concentrations. The formation of ice can also reduce the efficiency or even the structural integrity of inlet and outlet structures. The mine operators have experienced damage to drop inlet spillways resulting from ice loads, and consequently all their ponds are now constructed with simple CSP culvert installations that are not as subject to ice damage.

2.9.1.2.4 Cost. Unfortunately capital cost data for the complete pond B installation including chemical feed facilities were not available. Annual operating costs, excluding any allowance for sludge disposal, would consist mainly of those charges associated with the maintenance of the chemical feed facilities. These charges would include polymer costs, energy costs, labor costs, and the cost of bringing sufficient quantities of dilution water to site. Of these charges, the only item for which sufficient data was available for estimating purposes was polymer cost. It was estimated that the chemical costs for the pond B operation are in the vicinity of $20 000 per year (1985 $ Cdn).

2.9.1.2.5 Discussion. One of the reasons this particular pond was selected for a case history is that the methodology used for its design is fairly representative of what could be described as current practice in Alberta. In this methodology a certain particle 'cut size' is selected and the pond is then sized to remove all particles larger than this diameter. It is also commonly assumed that, if necessary, particles smaller than this diameter can be flocculated with polymers into more settleable sizes. There are several inadequacies associated with this approach that are described in some detail in Section 4.1.2, however pond B appears to be an example of a fairly successful application of this methodology. This conclusion regarding pond B is based on the following observations:

1. The pond achieves high sediment removal efficiencies and has a relatively good record of compliance;
2. The chemical dosages required to maintain this compliance record are fairly moderate; and
3. The utilization of available sediment storage is consistent with the expected service life of the pond (i.e., storage capacity will be exhausted at about the same time that the pond is decommissioned).

2.9.2 Reactor Models

The reactor model case history was derived from a settling pond performance evaluation conducted at a surface mine that will be referred to hereafter as mine B. Mine B is an open pit coal mine situated in the rolling prairie region east of the foothills in central Alberta that produces thermal coal for a local power generation facility.

The mine leases are located in the watershed of a significant regional lake that supports a high level of recreational activity. The operation encompasses a number of independent pits with relatively long lifespans (i.e., 15 to 20 years), which impact both the regional groundwater flows and numerous small streams in the area. Topographic relief over the site is typically rolling terrain. Prior to the initiation of mining the land was either forested or supported a variety of agricultural uses.

Due to a proposed modification to the mine water management plan for one of the pits, an evaluation of the impact of introducing additional flows to an existing settling pond installation was required. It was decided to use the CSTRS reactor model within the framework of the SEDIMOT II hydrology-sedimentology computer program to complete the evaluation. The original objective of this pond evaluation, which Monenco undertook for the pond operators, was to estimate the existing reservoir's performance under average controlled (pumped) flow conditions rather than 1 in 10 year storm conditions. Considerable work was expended in developing inputs and running the computer model before it was concluded that the present SEDIMOT II format could not be used to model this flow condition. This was because the current SEDIMOT II format does not allow the watershed component to be separated from the pond component. Since the program is a single storm event model it was therefore not possible to input long-term, controlled flows directly. Efforts to approximate these flows by manipulating the storm data were reasonably successful but it was subsequently found that the program would not allow these relatively low flows to be routed through the impoundment for more than 50 hours. Since the settling ponds had a storage volume greater than the 50-hour controlled flow volume, the flow could not be routed long enough for steady state effluent concentrations to be reached.

Because of the above difficulties, Monenco's original modelling efforts for the pond operators were abandoned in favour of the traditional USEPA overflow rate approach. For the purposes of this document, however, it was decided to make use of the work that had been expended on setting up the model by re-running the program for the 1 in 10 year storm flow condition. Since it was not possible to calibrate the model's input parameters with the particulars of the watershed to the extent that would be required for definitive evaluations of pond performance, the model outputs summarized in subsequent sections should be interpreted as being appropriate for illustrative purposes only. They do not necessarily reflect actual conditions or performance that could be expected during 1 in 10 year storm conditions.

2.9.2.1 Disturbed area description. The disturbed area in question consists of approximately 400 ha of gently undulating to rolling terrain, which prior to mining was used as pasture. The surficial geology in the area can be summarized as consisting of a shallow glacial till blanket overlying mudstones, siltstones, and shales. The coal is extracted using area-mining techniques in which the overburden is removed down to southwest dipping coal seams. The pit intersects both surficial and groundwater flows and has various areas currently undergoing prestripping, active mining, or reclamation. The pit configuration is shown in Figure 41.

The pit water management plan incorporates the use of diversion ditches, pit water effluent ditches, pit pump dewatering, and sedimentation ponds. The diversion ditches to the south of the active pit divert surficial flows from the upland areas around the pit to the lake. The east effluent ditch prevents mine water from entering an adjacent natural stream while the north effluent ditch prevents mine runoff from entering the lake. Both effluent ditches discharge directly to the sedimentation ponds. Pit floors receive both intercepted groundwater and upland runoff from the areas between the pit and the diversion ditches. All pit water is collected in sumps and pumped to the settling ponds.

Due to the dip of the coal seams being mined, the pit floor drains both east and west from a central divide. In the original pit water management plan, pit waters collected on the east side were to be treated at the existing east ponds prior to being discharged to the lake. West pit waters were to be treated at an additional set of ponds constructed north of the west pit area. These west pit ponds were not constructed initially because mining was to start on the east side and continue there for some time before operations were initiated on the west side. A proposed modification to the pit water management plan included elimination of the unbuilt west ponds and the diversion of all west mine waters to the existing east settling ponds. The impact that these additional flows would have on existing east pond effluent quality was uncertain and therefore it was decided that it would be necessary to estimate that quality prior to effecting the mine water diversion.

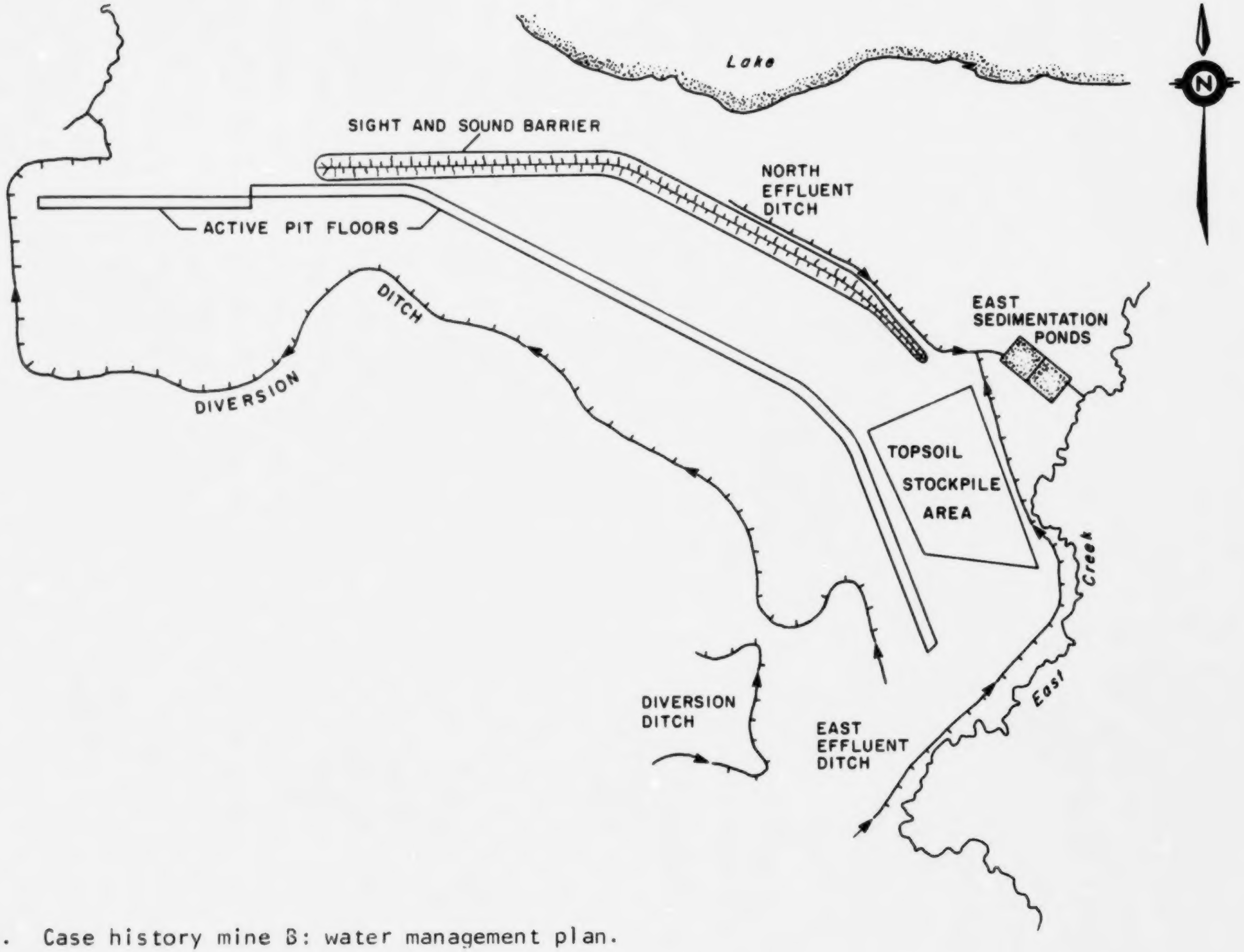

Lake
N
SIGHT AND SOUND BARRIER
NORTH EFFLUENT DITCH
ACTIVE PIT FLOORS
EAST SEDIMENTATION PONDS
DITCH
DIVERSION
TOPSOIL STOCKPILE AREA
Creek
East
DIVERSION DITCH
EAST EFFLUENT DITCH

Figure 41. Case history mine B: water management plan.

2.9.2.2 Settling ponds

2.9.2.2.1 Description. The east pit settling reservoir consists of two separate ponds in series and is shown in Figure 42. Mine water enters the pond via the two effluent ditches and is discharged from both the primary and secondary cells using clusters of four 400 mm diameter corrugated steel drop inlet spillways. Reservoir effluent is routed through a ditch to a nearby creek, which eventually discharges to the lake.

The inlet pond has a total depth of 2.20 metres and a high water elevation defined by a drop inlet spillway height of 1.45 m above the pond bottom. Under conditions of zero inflow, two holes in each spillway barrel slowly dewater the pond to a depth of 0.9 m. At the high water level the pond has a total surface area of 1.78 ha, while at the 'dewatered' elevation it has a surface area of 1.69 ha.

The outlet pond is 1.95 m deep and has drop inlet spillways at a height of 1.15 m above the pond bottom. The spillways eventually dewater the pond to a depth of 0.6 m. The total available surface areas at both water elevations are the same as those for the inlet pond.

2.9.2.2.2 Original design criteria. The ponds were originally designed using the USEPA overflow rate approach with a non-ideal settling factor of 1.67 (as opposed to the typical factor of 1.2) and a peak flow resulting from the 1 in 10 year, 24-hour storm event of 0.5 m^3/s. It should be noted that virtually all of this flow was assumed to be controlled (i.e., pumped) flow from the pit. The ponds were sized to provide a minimum detention time of 24 hours and to effect the complete removal of all particles larger than 10 microns in diameter at a water temperature of 10°C. It was assumed that if large concentrations of smaller particles were encountered, polymer flocculants could be used to bring the effluent into compliance with the requirements of the 'Alberta Coal Mining Waste Water Effluent Guidelines' (see Table 1).

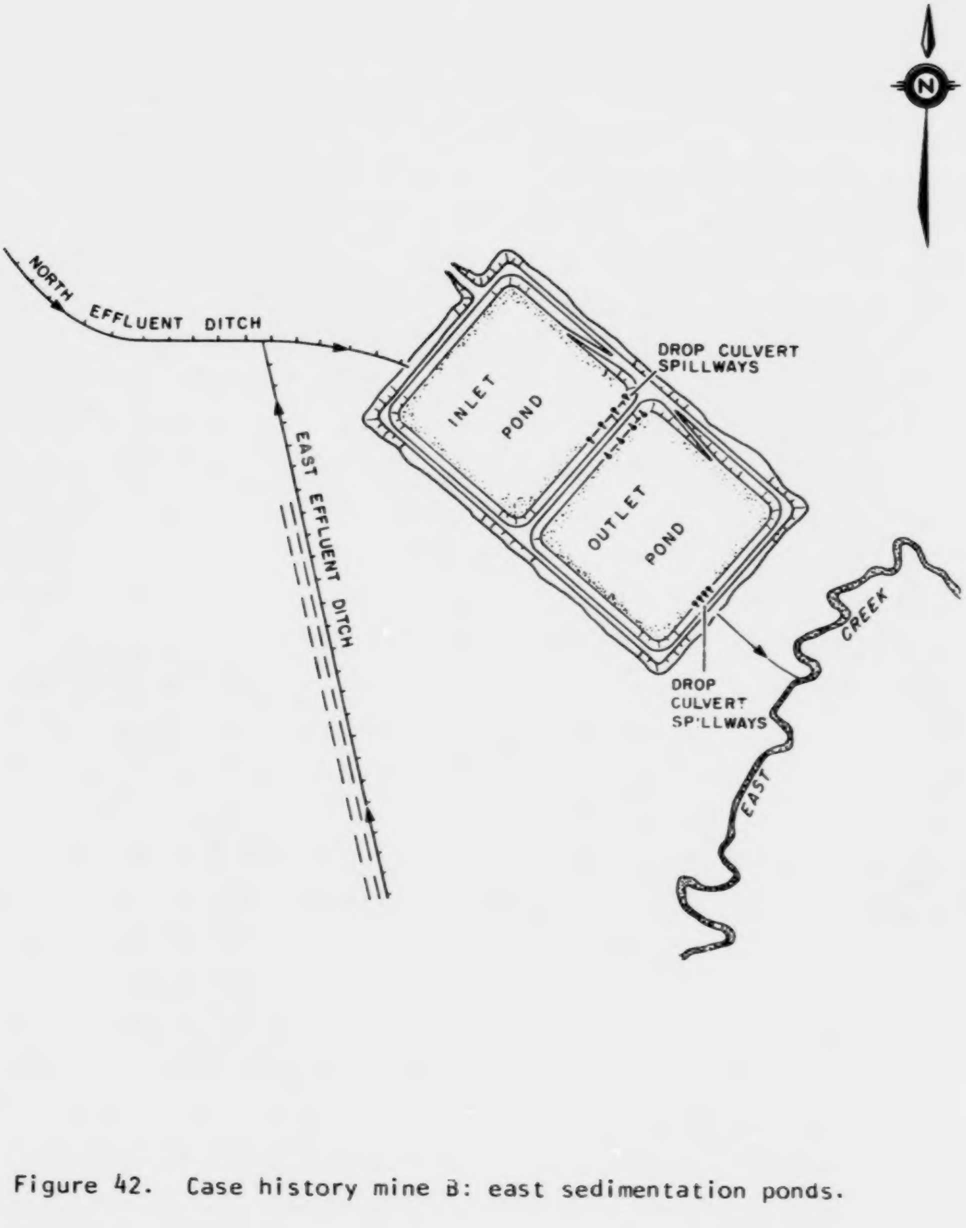

NORTH EFFLUENT DITCH
EAST EFFLUENT DITCH
INLET POND
OUTLET POND
DROP CULVERT SPILLWAYS
DROP CULVERT SPILLWAYS
EAST CREEK
N

Figure 42. Case history mine B: east sedimentation ponds.

2.9.2.2.3 Cost. The settling ponds were constructed using local materials for embankment construction and were commissioned in 1983. The total cost of the reservoir, including inlet and outlet structures and fencing, was approximately $280 000 (1985 $ Cdn).

2.9.2.3 Design model development and application. The CSTRS subroutine of the Sedimot II computer watershed model was used to evaluate the pond performance during a 1 in 10 year, 24-hour storm. For this storm event the pond inflow would consist of uncontrolled flow from the effluent ditches combined with controlled flows from the pit sumps. Since the volume of these storm flows would be very much greater than groundwater flows during the same period, the analysis neglected the effects of groundwater flow. As discussed previously in Section 2.5.3.2.1, the large storm flows draining into the pit and the limited pumping capacity available for removing them result in a significant period of in-pit detention for the mine runoff. This additional detention time was included in the evaluation by modelling the in-pit storage with a number of single cell CSTRS settling basins. The discharge rate from these 'pseudo' settling ponds was then set in accordance with the expected capacity of the pumping facilities. This simulated pit pumping arrangement is shown in Figure 43.

The size of the contributing watershed, and the variety of land uses that it supported, necessitated that it be divided into a number of subwatersheds for the definition of the computer model inputs. The subwatersheds used are shown in Figure 44.

2.9.2.3.1 Model inputs. The model inputs used in the computer program are summarized in Table 40. While some of these inputs are based on site-specific information derived from operating data for the existing east settling ponds, other program inputs (e.g., CN numbers) were developed from standard references (e.g., Tables 2 and 3).

2.9.2.3.2 Model outputs. The computer model outputs for the 1 in 10 year, 24-hour storm are summarized in Tables 41 to 43 and

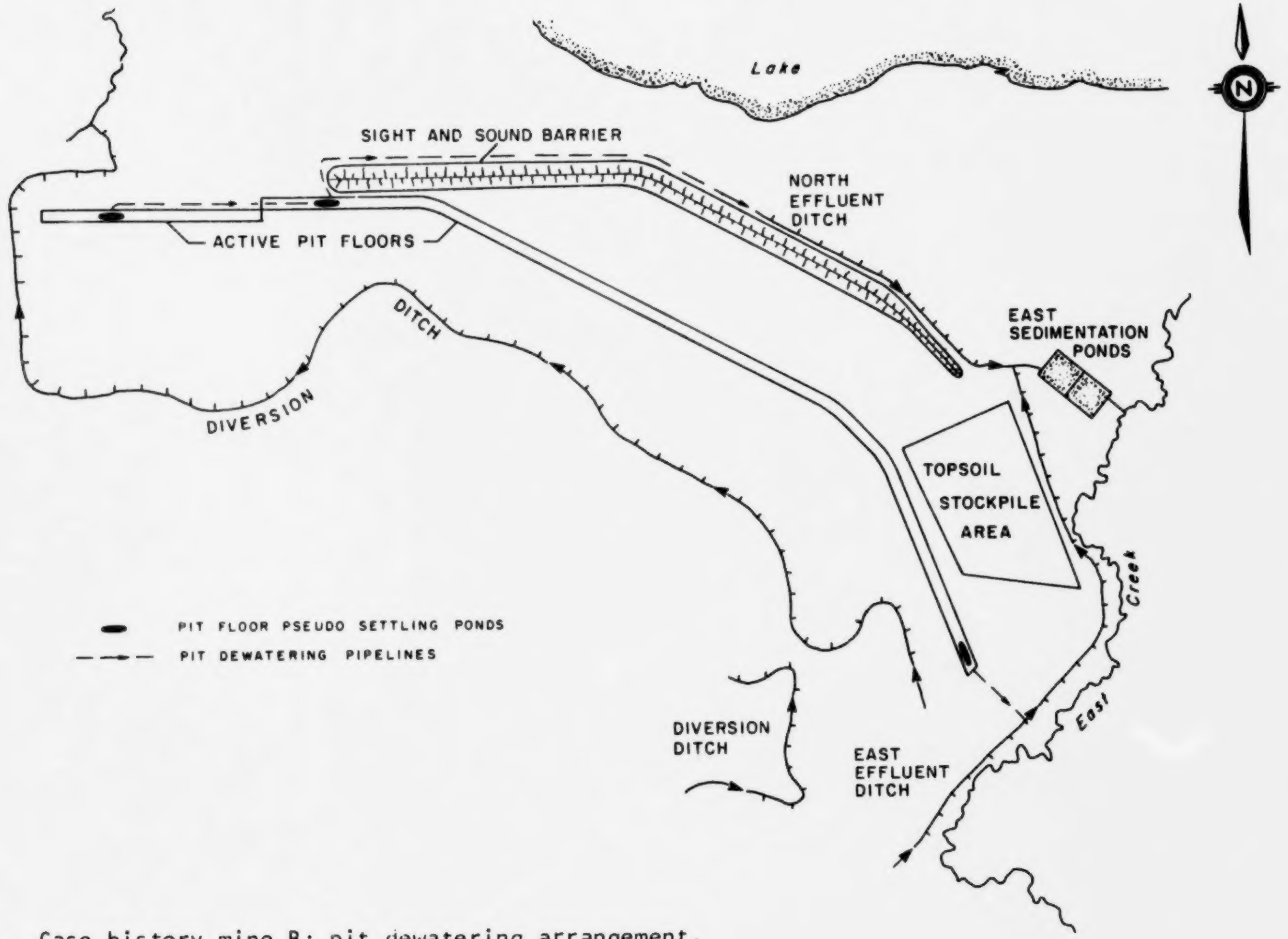

Lake
N
SIGHT AND SOUND BARRIER
NORTH EFFLUENT DITCH
ACTIVE PIT FLOORS
EAST SEDIMENTATION PONDS
DITCH
DIVERSION
TOPSOIL STOCKPILE AREA
Creek
East
DIVERSION DITCH
EAST EFFLUENT DITCH
PIT FLOOR PSEUDO SETTLING PONDS
PIT DEWATERING PIPELINES

Figure 43. Case history mine B: pit dewatering arrangement.

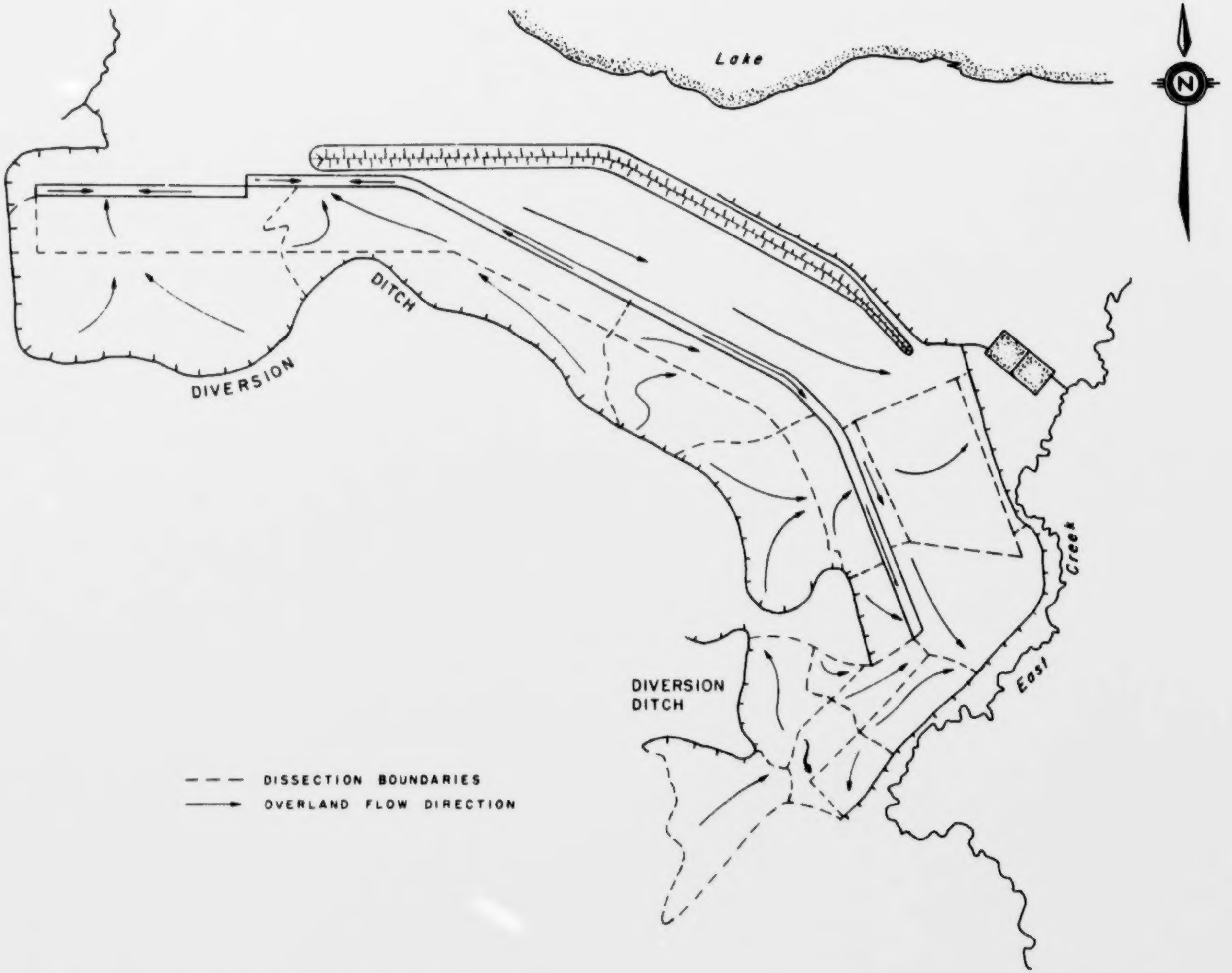

Lake
N
DITCH
DIVERSION
DIVERSION
DITCH
East
Creek
DISSECTION BOUNDARIES
OVERLAND FLOW DIRECTION

Figure 44. Case history mine B: model watershed dissection.

Table 40. Mine B case history: computer model input data.

1. Storm information

- Frequency = 10 years
- Duration = 24 hours
- Total precipitation = 74 mm
- Temporal rainfall pattern = SCS Type II

2. Sediment characteristics

- Specific gravity = 1.69
- Submerged bulk specific gravity = 1.05
- Particle size distribution:[a]

Size (microns)	% Finer	
	Disturbed Areas	Undisturbed Areas
100	100.0	100.0
90	99.5	90.0
80	99.0	79.0
70	98.5	70.0
60	98.0	61.0
45	97.5	49.5
30	96.0	39.0
20	93.5	30.0
12	88.5	19.0
8	83.0	12.0
5	70.0	7.0
4	63.0	6.0
3	52.0	5.0
2	33.0	3.0
1	0.0	0.0

continued...

Table 40. Concluded.

- Load rate exponent factor = 2.0

3. Watershed characteristics

	Land Use				
	Natural Pasture	Prestripped Areas	Levelled Spoil	Pit Floors	Stockpiles
- CN[b]	75	85	91	91	30
- Runoff Hydrograph Response	Undisturbed	Disturbed	Disturbed	Disturbed	Disturbed
- K[c]	0.30	0.35	0.25	0.25	0.35
- CP[d]	0.20	1.20	1.20	1.20	0.80
- Particle Size Distribution	Undisturbed	Disturbed	Disturbed	Disturbed	Undisturbed

- Slope length, slope gradient, area, time of concentration, and channel routing coefficients were determined for each subwatershed individually using topographic information

---

[a] This is the particle size distribution of sediment immediately after it is eroded. The program adjusts this distribution as it is routed through the watershed to arrive at the pond influent particle size distribution.

[b] CN = runoff curve number

[c] K = soil erodibility factor

[d] CP = control practice factor

Table 41. Mine B case history; summary of inlet pond influent characteristics.

Total Sediment Yield = 2011 Tons

Total Runoff Volume = 113 Acre-Ft

Peak Runoff Discharge = 242 CFS

Total Drainage Area = 1011 Acres

Peak Concentration = 67276 MG/L

Peak Settleable Concentration = 4.9 ML/L

Peak Settleable Concentration = 5134. MG/L

Period of Significant Concentration = 44.1 Hrs

Volume Weighted Average Settleable Concentration During Period of Significant Concentration = 1.6 ML/L

Volume Weighted Average Settleable Concentration During Peak 24-Hour Period = 1.8 ML/L

Arithmetic Average Settleable Concentration During Period of Significant Concentration = 0.5 ML/L

Arithmetic Average Settleable Concentration During Peak 24-Hour Period = 0.7 ML/L

continued...

Table 41. Concluded.

Sediment particle size distribution:

Size (microns)	% Finer
100	100
90	100
80	100
70	100
60	100
45	100
30	99.1
20	97.2
12	93.4
8	89.1
5	80.2
4	75.2
3	65.0
2	43.0
1	0.4

Table 42. Mine B case history; summary of inlet pond outlet characteristics.

Turbulence Factor	=	1.0	
Permanent Pool Capacity	=	11.84	Acre-Ft
Dead Storage	=	24.0	Percent
Time Increment Outflow	=	0.10	Hrs
Viscosity	=	0.009	CM**2/SEC
Inflow Runoff Volume	=	112.8	Acre-Ft
Outflow Routed Volume	=	99.5	Acre-Ft
Storm Volume Discharged (Plug Flow)	=	99.5	Acre-Ft
Pond Volume at Peak Stage	=	43.9	Acre-Ft
Peak Stage	=	10.0	Ft
Peak Inflow Rate	=	242	CFS
Peak Discharge Rate	=	50.6	CFS
Peak Inflow Sediment Concentration	=	67276.	MG/L
Peak Effluent Sediment Concentration	=	29373.	MG/L
Peak Effluent Settleable Concentration	=	0.01	ML/L
Peak Effluent Settleable Concentration	=	10.3	MG/L
Storm Average Effluent Concentration	=	21073	MG/L
Average Effluent Sediment Concentration	=	21073	MG/L
Basin Trap Efficiency	=	19.8	Percent
Detention Time of Flow with Sediment	=	4.8	Hrs
Detention Time From Hydrograph Centers	=	4.8	Hrs
Detention Time Including Stored Flow	=	4.8	Hrs
Sediment Load Discharged	=	1614	Tons
Period of Significant Concentration	=	40.1	Hrs
Volume Weighted Average Settleable Concentration During Period of Significant Concentration	=	0.01	ML/L
Volume Weighted Average Settleable Concentration During Peak 24-Hour Period	=	0.01	ML/L

continued...

Table 42. Concluded.

Arithmetic Average Settleable Concentration During Period of Significant Concentration	=	0.00	ML/L
Arithmetic Average Settleable Concentration During Peak 24-Hour Period		0.01	ML/L

Sediment particle size distribution:

Size (microns)	% Finer
100	100
90	100
80	100
70	100
60	100
45	100
30	100
20	100
12	100
8	100
5	99.9
4	93.7
3	81.0
2	53.6
1	0.5

Table 43. Mine B case history; summary of outlet pond outlet characteristics.

Turbulence Factor	=	1.0	
Permanent Pool Capacity	=	8.2	Acre-Ft
Dead Storage	=	31.0	Percent
Time Increment Outflow	=	0.10	Hrs
Viscosity	=	0.009	CM**2/SEC
Inflow Runoff Volume	=	112.8	Acre-Ft
Outflow Routed Volume	=	97.4	Acre-Ft
Storm Volume Discharged (Plug Flow)	=	97.4	Acre-Ft
Pond Volume at Peak Stage	=	23.9	Acre-Ft
Peak Stage	=	5.6	Ft
Peak Inflow Rate	=	50.6	CFS
Peak Discharge Rate	=	38.2	CFS
Peak Inflow Sediment Concentration	=	29373	MG/L
Peak Effluent Sediment Concentration	=	25896	MG/L
Peak Effluent Settleable Concentration	=	0.00	ML/L
Peak Effluent Settleable Concentration	=	1.3	MG/L
Storm Average Effluent Concentration	=	19893	MG/L
Average Effluent Sediment Concentration	=	19893	MG/L
Basin Trap Efficiency	=	18.5	Percent
Detention Time of Flow with Sediment	=	3.3	Hrs
Detention Time From Hydrograph Centers	=	3.3	Hrs
Detention Time Including Stored Flow	=	3.3	Hrs
Sediment Load Discharged	=	1314	Tons
Period of Significant Concentration	=	37.0	Hrs
Volume Weighted Average Settleable Concentration During Period of Significant Concentration	=	0.0	ML/L
Volume Weighted Average Settleable Concentration During Peak 24-Hour Period	=	0.0	ML/L
Arithmetic Average Settleable Concentration During Period of Significant Concentration	=	0.0	ML/L

continued...

Table 43. Concluded.

Arithmetic Average Settleable Concentration During Peak 24-Hour Period = 0.0 ML/L

Sediment particle size distribution:

Size (microns)	% Finer
100	100
90	100
80	100
70	100
60	100
45	100
30	100
20	100
12	100
8	100
5	100
4	100
3	99.4
2	65.8
1	0.6

Figures 45 to 51. The tables characterize both the influent entering the reservoir and the effluent discharged from the inlet and outlet settling pond cells. The figures show the inlet and outlet hydrographs and sedimentgraphs as well as the variation in particle size distribution as the mine drainage is routed through the reservoir. In all of the hydrographs and sedimentgraphs the differences in timing and the relative impacts on effluent quality of the controlled and uncontrolled portions of the total storm flow are well illustrated. The uncontrolled flow is represented in all of the graphs by the fairly sharp initial peak in both flow rates and sediment concentrations. The controlled flow impacts, by contrast, occur later, are of considerably lower magnitude, and continue over a much longer period of time. The figures show that for small to moderate size ponds, short duration, high intensity flows have a greater negative impact on effluent TSS concentrations than longer duration, low intensity flows of the same total volume. The last plot, Figure 51, illustrates how relatively coarse materials are the first to be removed in a settling pond. As the mine drainage moves through the reservoir, more and more coarse particles are removed, shifting the gradation to the finer end of the scale as ever-increasing proportions of the total solids content is made up of relatively small diameter particles.

2.9.2.4 Discussion. The SEDIMOT II computer program showed that the east settling pond effluent would be unlikely to comply with regulatory requirements during a 1 in 10 year storm event if west pit flows were added to the existing east pit flows. This inadequate performance would primarily be the result of the very fine eroded sediment particle size distributions known to be characteristic of disturbed soils in this area. The computer evaluation also showed that the inlet pond spillway structures would be incapable of passing the combined east and west pit flows during the design storm event. This can be seen from the contents of Table 42, which shows a peak inlet pond stage of 10.0 feet (3.0 metres), which is greater than the total available pond depth of 7.2 feet (2.2 metres).

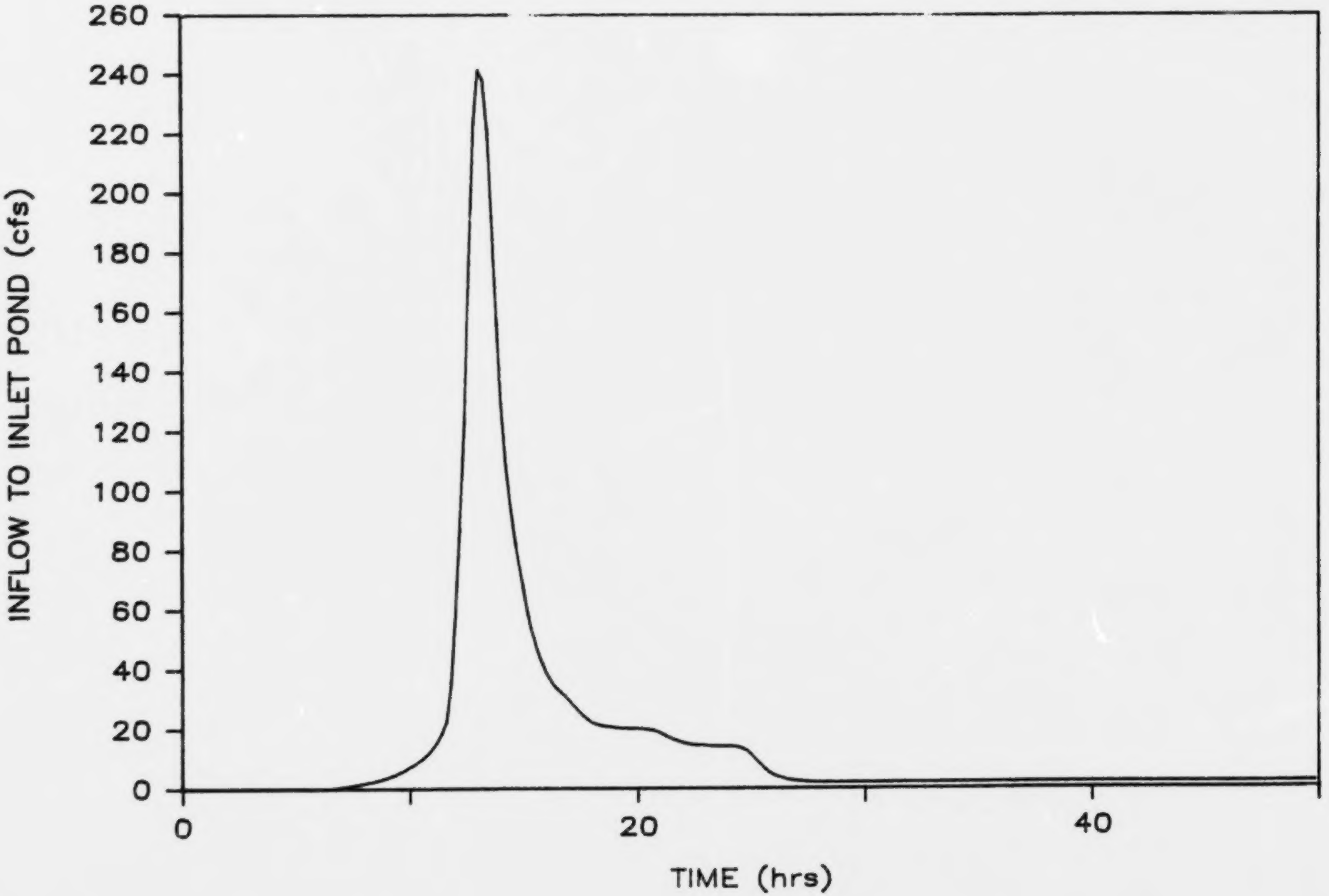


Figure 45. .e history mine B: predicted inflow to inlet pond.

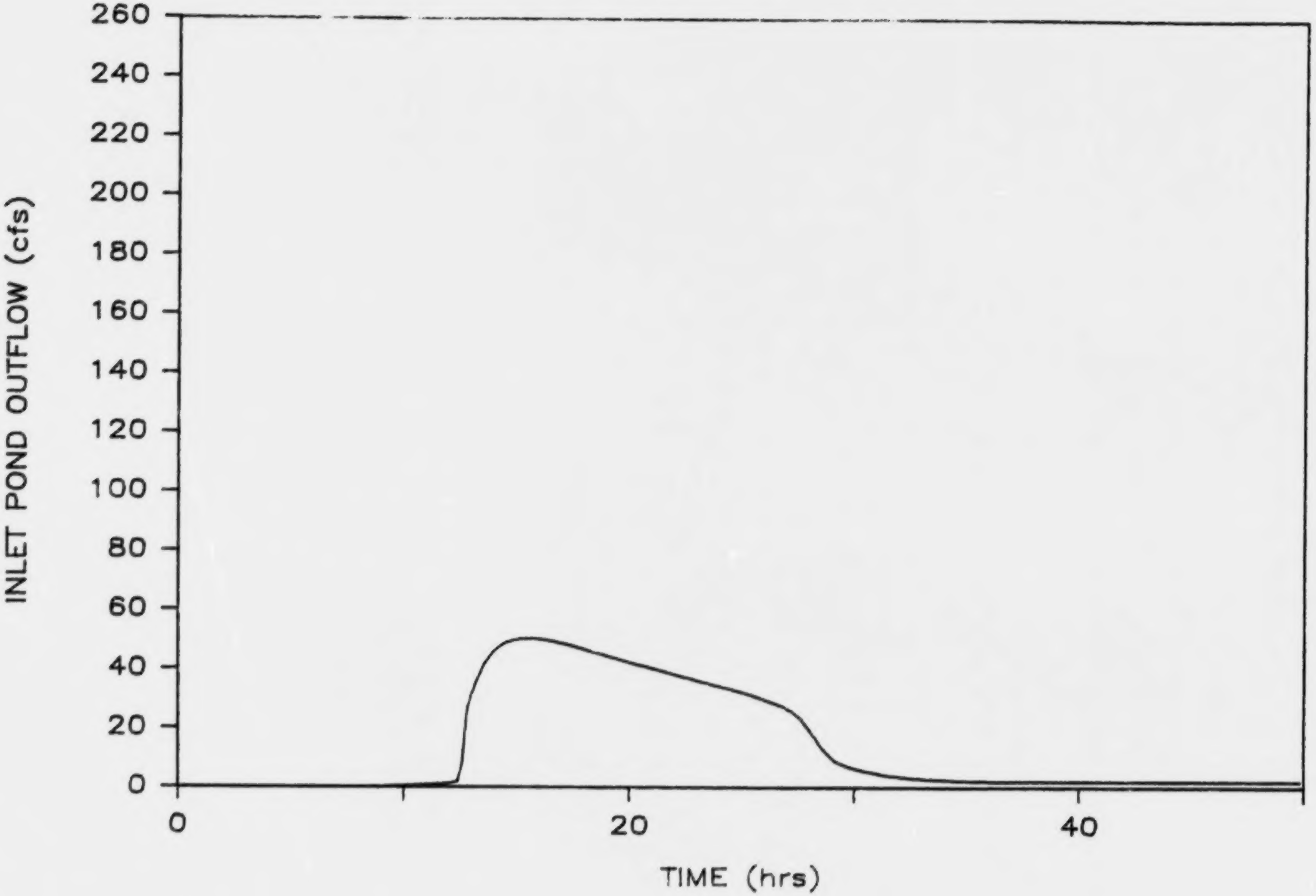


Figure 4b. Case history mine B: predicted inlet pond outflows.

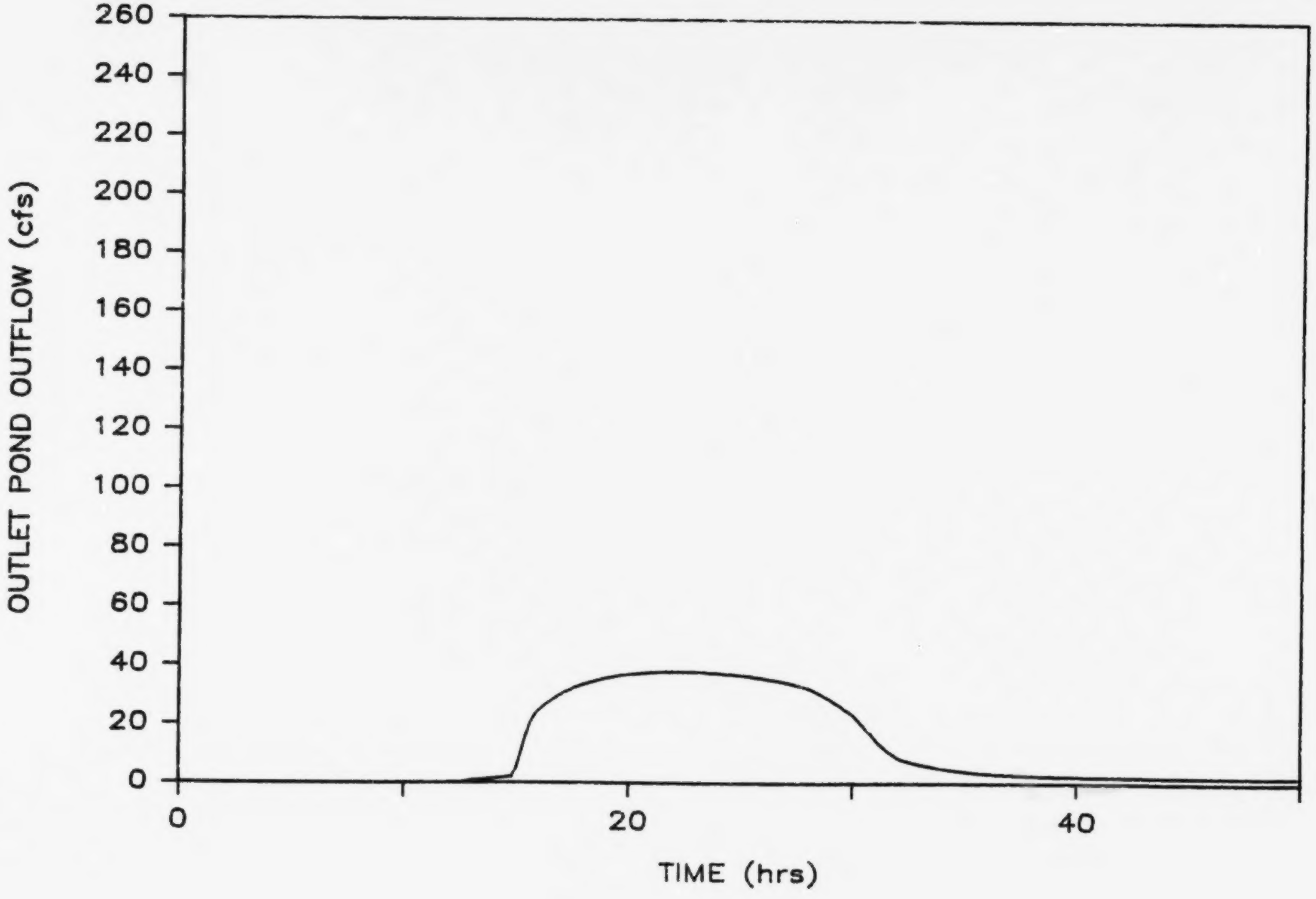


Figure 47. Case history mine B: predicted outlet pond outflows.

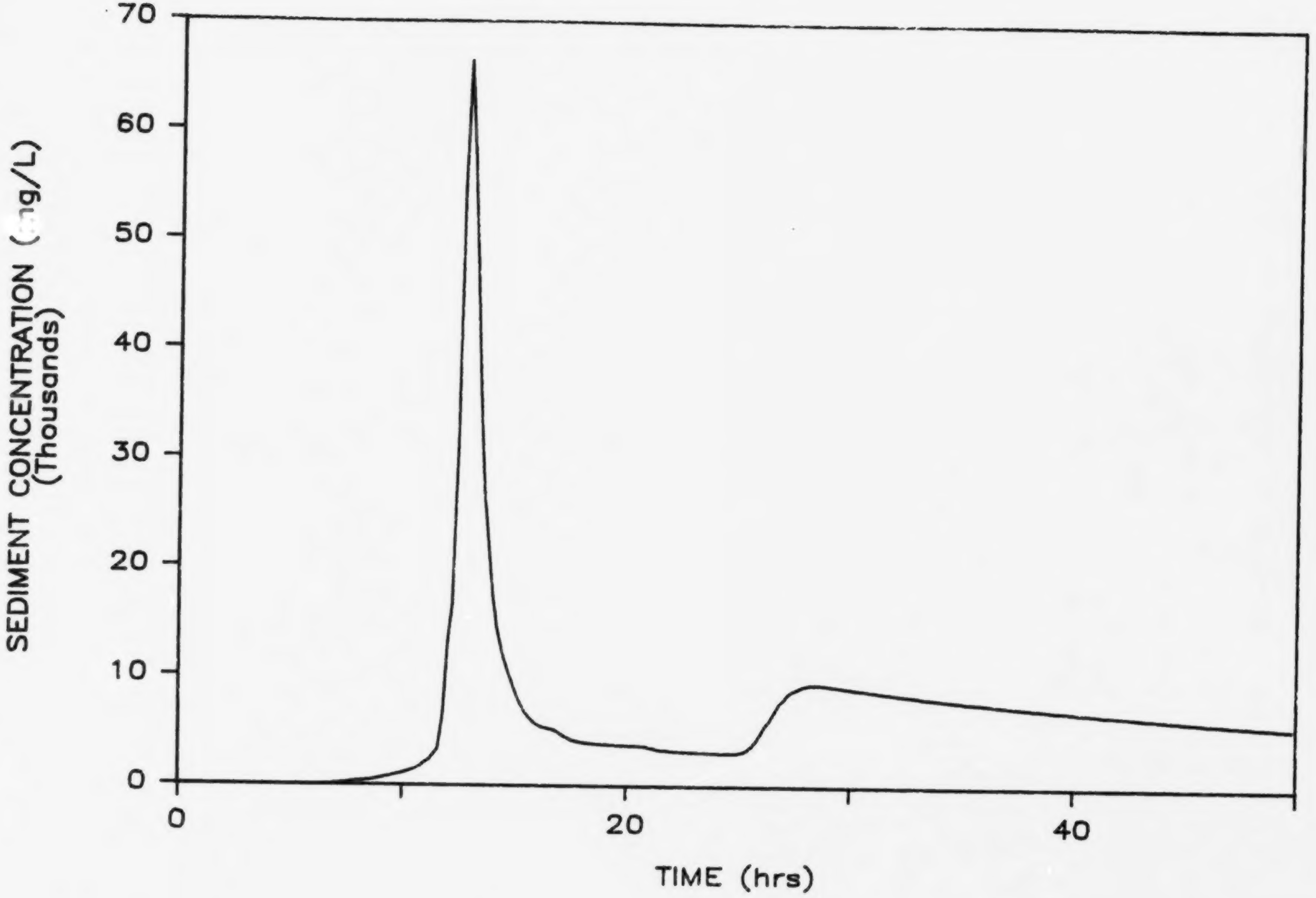


Figure 48. Case history mine B: predicted inlet pond influent TSS.

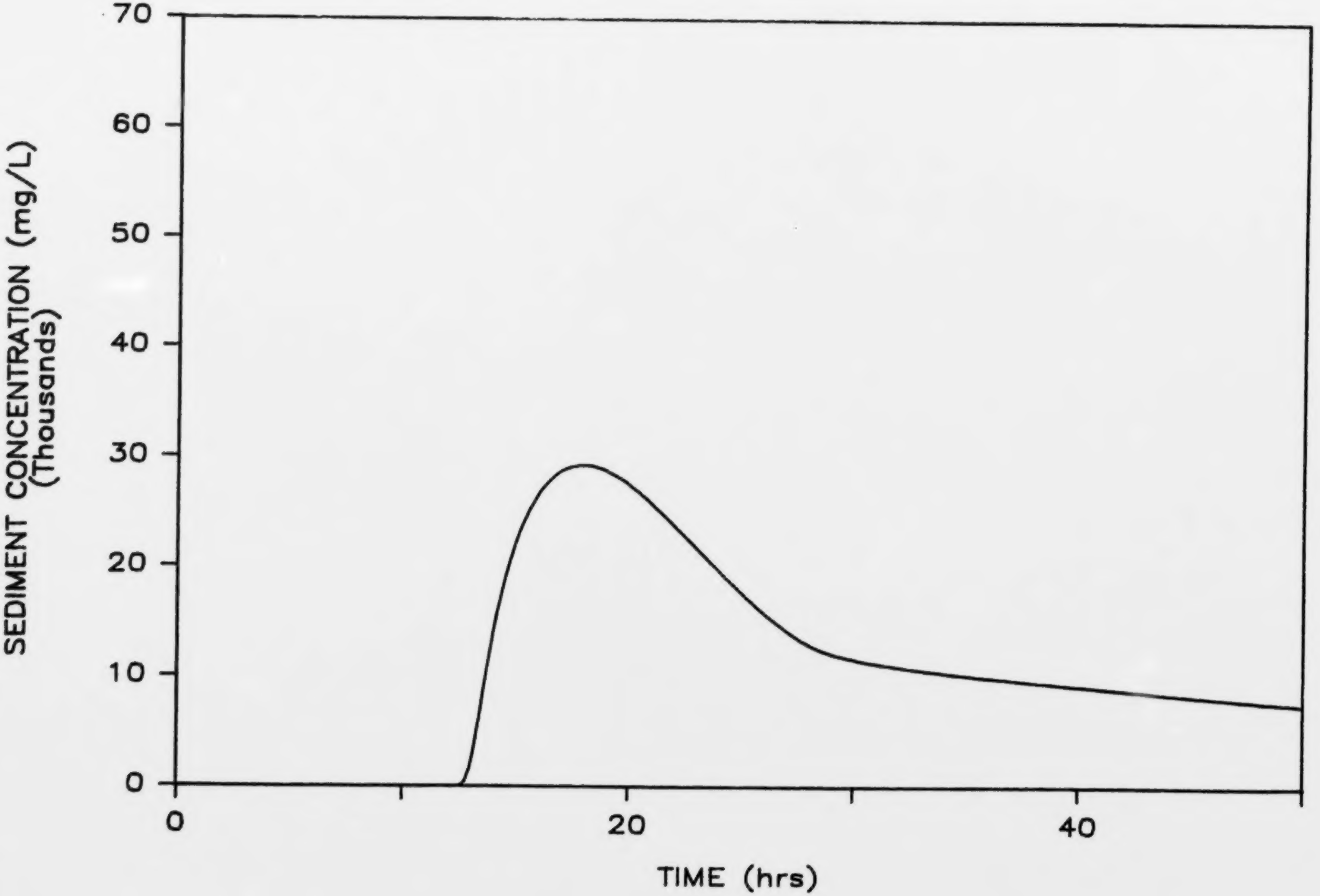


Figure 49. Case history mine B: predicted inlet pond outflow TSS.

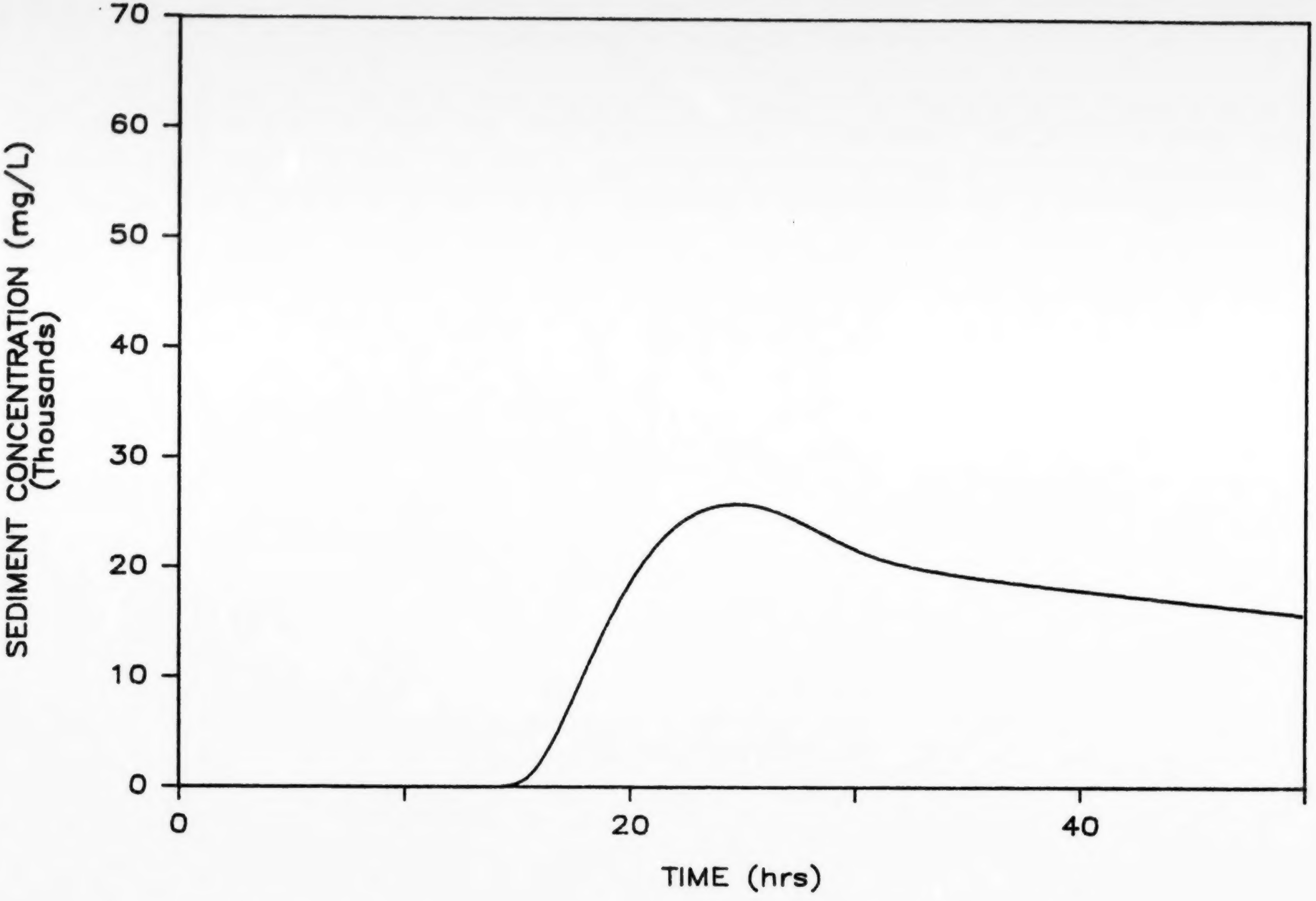


Figure 50. Case history mine B: predicted outlet pond outflow TSS.

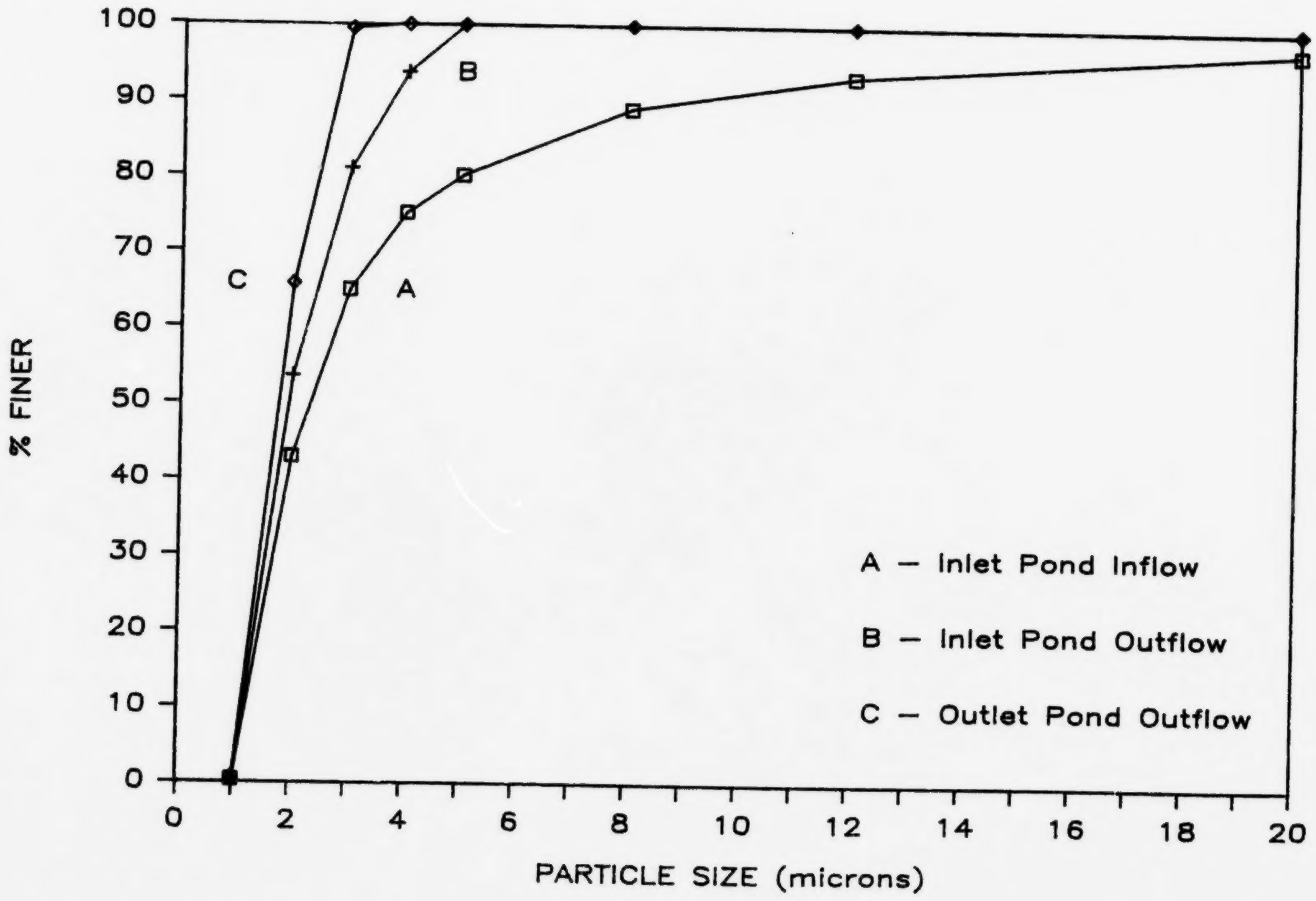


Figure 51. Case history mine B: predicted particle size distributions.

In addition to those inadequacies noted previously, the pond performance evaluation showed that SEDIMOT II does not have the capability to successfully simulate pumped pipeline flows. The program effectively assumes that piped flows travel between pipeline terminal points instantaneously. This can lead to serious errors in flow determinations when combining hydrographs based in part on piped flows for which actual travel times are significant. For this particular pond evaluation, piped flows were relatively low and had short travel times so that the error introduced by this inadequacy was not significant. However, for controlled flow operations using relatively long lengths of pipe, the inaccuracy introduced could be substantial.

It should be noted that the above difficulty, as well as those discussed earlier, are primarily associated with program format rather than content. In all probability these problems could be addressed with relatively minor modifications to the program. For example, the problems associated with modelling controlled flows could be resolved simply by adding the capability to use the program's watershed and pond components independently.

2.10 MAINTENANCE CONSIDERATIONS: SEDIMENT REMOVAL

The removal of accumulated sediments represents one of the predominant operating and maintenance considerations associated with settling ponds. When the sediment storage capacity of a pond has been exhausted, the sediments, or sludge, must be removed if the impoundment's solids removal capacity is to be maintained. The frequency of sediment removal and the scope of the operation required to affect that removal are directly related to the density of the in-situ sludge. Coarse-grained sediments consisting of large aggregates can be expected to exhibit relatively high sludge densities and correspondingly low sediment removal frequencies. On the other hand, very fine-grained sediments require frequent sludge removal and provisions for handling and disposing of relatively large volumes of dilute sludge. The entire sediment removal operation can be split into three basic phases:

1. Excavation,
2. Handling/dewatering, and
3. Disposal.

The following discussion briefly describes some of the considerations associated with each of these phases.

2.10.1 Excavation

There are a number of methods available for excavating pond sediments. The method chosen for a particular location would depend on the density of the sludge to be removed, the size of the basin, and the availability of suitable equipment.

2.10.1.1 Hydraulic dredges. Small hydraulic dredges are often used for the removal of relatively low density sludges. These dredges normally consist of sludge pumps mounted on pontoons so that the entire assembly can easily be moved around on an impoundment. A typical hydraulic dredge of this type is shown in Figure 52. Resuspension of sediment during dredging is kept to a minimum by maintaining the pump intake well below the sludge/supernatant interface so that the flow pattern is always from the effluent into the sludge.

2.10.1.2 Conventional draglines. A commonly used method of cleaning ponds involves the use of a crane with a dragline bucket attached. The crane is positioned on the pond bank and then its bucket is 'thrown' into the water by a rapid rotation of its boom. As the bucket is drawn across the pond it scrapes a load of sediment from the bottom. The dredged material is either piled in windrows along the pond bank or loaded directly onto a transport vehicle.

The reach of a dragline suitable for this kind of application is typically limited to about 20 metres, so that this method of excavation is normally used only for settling ponds less than 40 metres across. Since the dragline is positioned on the edge of a

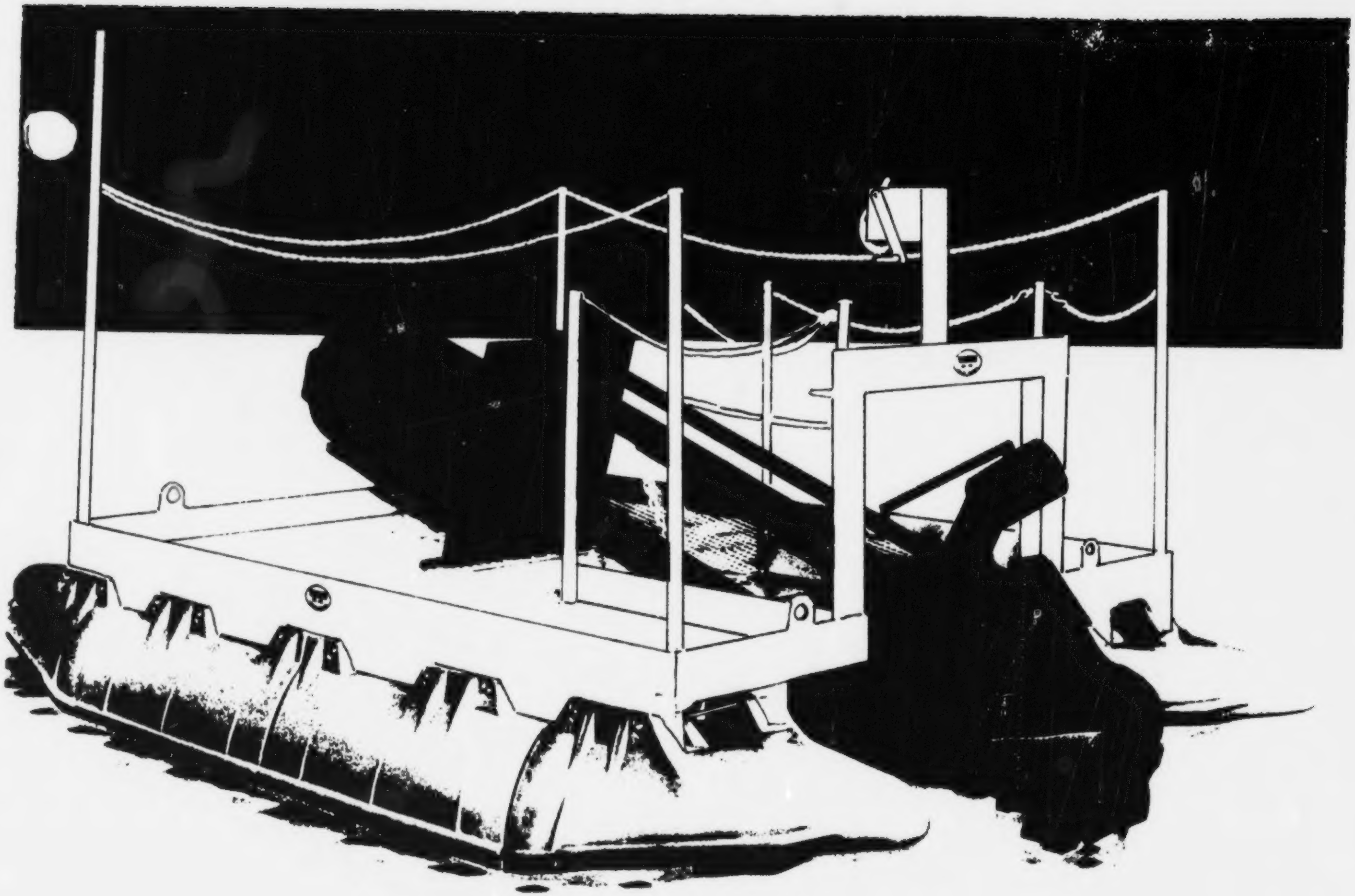


Figure 52. Small hydraulic dredge.(National Car Rental,undated)

settling pond, this method offers an advantage for situations in which the pond bottom is too soft to support other types of cleaning equipment (Thronson 1976).

2.10.1.3 Crane-operated scrapers. Excavation using crane-operated scrapers is similar to the conventional dragline approach except that the effective reach of the operation is extended to about 300 metres. Two of the most common approaches used in this method are illustrated in Figures 53 and 54. As shown in the figures, scrapers are hauled in by cranes or winches and returned by a cable attached to a 'deadman', or support, on the opposite pond bank. Larger scrapers can be used and greater mobility achieved by using a boom support for the crane and a bulldozer for the tail anchor as shown in Figure 54. Both of the methods illustrated are similar to conventional draglines in that they offer the advantage of being suitable for applications with soft pond bottoms (Thronson 1976).

2.10.1.4 Front-end loaders. For ponds with relatively high sludge densities, conventional front-end loaders are often used for sediment removal. A disadvantage of this approach is that a pond must be removed from service and drained prior to the initiation of excavation operations. However, for situations where temporary pond downtime is acceptable, this form of sludge removal is usually the most competitive in terms of cost (Thronson 1976).

2.10.1.5 Reducing excavation costs. The cost of excavating deposited sediment can be reduced in some cases by constructing relatively small reservoirs upstream of the main settling pond(s). These impoundments remove most of the coarse-grained material in a small structure that can easily be excavated with conventional earth moving equipment. Although these smaller ponds will require more frequent cleanout, the unit costs of sediment removal may be lower since specialized equipment with a long reach is not required.

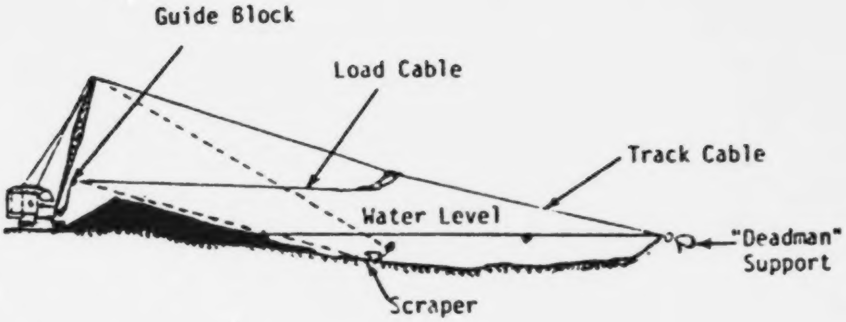

Guide Block
Load Cable
Track Cable
Water Level
"Deadman" Support
Scraper

Figure 53. Typical crane-operated scraper arrangement. (Thronson 1976)

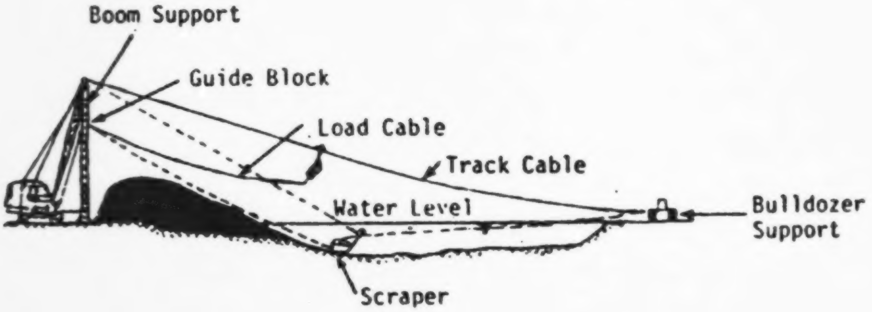

Boom Support
Guide Block
Load Cable
Track Cable
Water Level
Bulldozer Support
Scraper

Figure 54. High capacity crane-operated scraper arrangement. (Thronson 1976)

2.10.2 Handling/Dewatering. Once the sediment has been excavated from a pond it must be loaded onto some form of transport vehicle and delivered to the ultimate disposal site. For very dilute sludges, some form of dewatering is often required as an intermediate step. Sludge dewatering is usually accomplished using one of the following technologies:

1. Vacuum filtration,
2. Centrifugation,
3. Lagoon thickening,
4. Porous bed drying,
5. Sediment reworking, and
6. Sediment rehandling.

Filtration and centrifugation have been used for many years in a wide variety of sludge dewatering applications and are described in some detail in Section 3.2. Lagoon thickening is another very common dewatering technology which, if land is readily available, is often the most cost-effective thickening alternative. In this method, the dilute sludge is simply retained in a pond for a period of time sufficient to affect the desired increase in density. The pond supernatant is then decanted and the sludge either excavated immediately or allowed to dry if a still higher density is desired.

Porous drying beds consist of an impoundment underlain by a drained bed of highly permeable sand and gravel. The impounded sludge is gradually thickened as entrained water migrates toward the porous pond bottom. One problem with this approach is that the gravel bed eventually becomes clogged with sediment and must be replaced, an expensive and time-consuming task.

The dewatering characteristics of a sludge can be improved by reworking the top portion of sediments that have either been removed from a pond or remain in a pond after supernatant decanting. The kneading action applied by whatever mechanism is used to rework the sediment brings water to the surface, where it eventually evaporates. Experiments have shown that scarifications applied by a crane-operated scraper can effectively increase the density of the upper 0.3 metres of sludge (Thronson 1976).

It has also been found that simply rehandling excavated sediment can increase its density. If a dilute sludge is initially deposited in a contained area, a second excavation using a dragline or crane scraper is often enough to increase density to the point where containment is no longer required (i.e., sludge density is sufficient to provide it with an angle of repose) (Thronson 1976).

The final sludge density achievable with any of the aforementioned techniques is highly variable and depends on a number of site specific factors. However, broad ranges of possible densities can be estimated and are presented for some of the technologies in Table 44. Although this table was taken from a reference on acid mine drainage and therefore is not directly applicable to neutral wastewaters, the figures presented are consistent with Monenco's experience with non-acid coal mine drainage sludge generated by mines in the Alberta plains region.

2.10.3 Disposal. In the final phase of the sediment removal operation, excavated and dewatered sludge is transported to its ultimate disposal location. When selecting an appropriate location for disposal, the characteristics of the sludge must be considered as well as the economics of transporting the sediment. Although most sediment deposits are relatively innocuous, it is usually prudent to test the sludge for pH, electrical conductivity, soluble nitrate, total phosphorous, soluble ion, and heavy metal content to ensure that its disposal in a particular area will not contaminate groundwater supplies or reduce the productivity of native soils.

The most economical ultimate disposal location is invariably the original settling pond itself. If sediment loads are small or the duration of disturbed area activity is short, it is quite possible that pond cleanout will never be required, and the accumulated sediments can simply be left in the pond. If this is not possible, the usual second choice is to transport the material to an

Table 44. Mine drainage dewatering methods and final densities.[a]

Dewatering Method	Sludge Cake (% Solids by Weight)
Vacuum filtration	9 - 31
Precoat vacuum filtration	11 - 35
Porous drying beds	15 - 25
Centrifugation	8 - 64
Settling pond	0.5 - 5
Thickening	12 - 20

[a] Adapted from original table in Escher, Kicinski, and Ninesteel (1983).

abandoned pit within the disturbed area. A third option would be to mix dried sediments with soils in reclamation areas. For this last alternative, the characteristics of the sludge must be evaluated carefully to ensure that the combined soil is not toxic to the vegetation proposed for the area.

Sediment is most commonly transported to the ultimate disposal location by truck. Dilute sludges (those less than about 15% solids by weight) can be pumped into tanker trucks for disposal, while more concentrated sludges are normally transported using conventional dump trucks.

The costs associated with excavating sediment and transporting it to the final disposal location can be very significant for ponds with high sediment loads, fine-grained particles, and long lifespans. An appreciation for the magnitude of sludge disposal costs and the variation of those costs with sludge density can be derived from the figures presented in Section 3.2.9.

2.11 PARTICULARS OF THE ALBERTA FOOTHILLS AND FRONT RANGES AS THEY RELATE TO SEDIMENT POND DESIGN

2.11.1 Topography

The topography of many disturbed areas in the Alberta foothills and front ranges is often characterized by relatively steep slopes. These steep slopes affect settling pond design and performance in two ways: by influencing the shape and area under the inflow hydrograph and by limiting the space available for pond construction. High gradients increase the velocity of overland flow, which tends to increase the peak and decrease the duration of pond inflows resulting from storm events. The high velocities associated with steep slopes also reduce water infiltration, which has the effect of increasing the total volume of runoff delivered to a pond. For disturbed areas with uncontrolled flow conditions, the combined effect of these factors results in an increased pond detention volume requirement. For controlled flow conditions (i.e., those in which virtually all of the pond inflow consists of water pumped from a pit), the net effect of steep slopes is very limited. In this case, the higher peak flows and runoff volumes into the pit may affect the timing and duration of pump operations, but they will not influence settling pond performance.

The performance of a pond for uncontrolled flow conditions in the presence of high average slopes (i.e., those above about 6%) is best evaluated using one of the design models that accounts for the temporal variation in pond inflow during storm events (e.g., DEPOSITS, CSTRS, and BASIN reactor models). The use of a steady flow model (e.g., USEPA overflow rate approach) will overestimate pond requirements if based on peak flow and underestimate those requirements if based on average flow.

There is another feature of the traditional steady flow design models that tends to reduce their appropriateness for steeply sloped watersheds. These models are invariably derived from the simple overflow rate method, which predicts sediment removal efficiencies based solely on the surface area available for settling. This approach assumes that a particle must fall the full depth of the basin to the sludge bed if it is to be removed from solution. This assumption ignores the inherent advantage of the common surface withdrawal system for which a particle simply has to fall to a depth below the invert of the outlet structure to avoid being discharged with the effluent. When the vertical location of the discharge structure is considered, detention time and hence, basin depth, become very important. When design models that ignore detention time are applied to situations in which ponds are formed by damming natural valleys bounded by steep slopes, a very deep pond is often required to bring the water's surface to an elevation that will provide the necessary surface area. When these ponds are used with surface withdrawal systems, the resulting design is often overly conservative from the point of view of sediment removal efficiency because the advantages derived from the high detention time provided by the large depth have not been considered. In this sort of circumstance, one of the reactor models (DEPOSITS, CSTRS, or BASIN) would provide more economical estimates of sediment basin requirements.

The space available for settling pond construction in areas characterized by steep slopes is often very limited. In this case, the production of acceptable effluent quality may require a concerted effort to either reduce pond sediment loads or improve the efficiency of the maximum pond size available using one or more of the augmentative technologies described in Section 3.1. In extremely confined areas it may be necessary to consider using one of the 'stand alone' alternative technologies presented in Section 3.2.

2.11.2 Winter Operations. The severe winters experienced in many parts of the Alberta foothills and front ranges can have a substantial impact on settling pond performance that should be considered during the design phase. The following conditions are those encountered during winter operations that have the most significant effect on settling pond performance:

1. Low water temperature,
2. Ice formation,
3. Low inflows, and
4. Low ambient air temperatures.

2.11.2.1 Water temperature. The viscosity of water increases significantly as its temperature approaches 0°C. Since particle settling velocity is directly related to water viscosity (see Section 2.6.1.1.1), a drop in water temperature will reduce particle settling velocities. For example, a drop in water temperature from 20°C to 4°C reduces particle settling velocities by 36% simply by virtue of increased water viscosity (Clark, Viessman, and Hammer 1977). The effect that this kind of reduction in settling velocity can have on sediment pond performance is fairly obvious. For quiescent flow conditions, a 36% reduction in particle settling velocities requires a corresponding increase in available detention volume for the maintenance of a constant effluent quality. The effect of variations in water temperature on the performance of a particular pond can be evaluated with any of the physically based sedimentation models discussed in Section 2.6.

2.11.2.2 *Ice formation*. The formation of ice can affect settling pond performance by reducing the available pond detention volume and by creating serviceability problems at inlet and outlet structures. A significant depth of ice effectively removes a portion of a pond's available detention volume, resulting in a marked reduction in the pond's capability to remove suspended solids, particularly those at the finer end of the incoming particle size distribution. This reduction in sediment removal capacity can be estimated by using any of the sedimentation models presented in Section 2.6 with a pond detention volume or settling area reduced to reflect the volume lost to the maximum expected thickness of ice.

The accumulation of ice in and around inlet and discharge structures can also create problems. Small diameter inlet and outlet culverts can build up accumulations of ice during low and/or intermittent flow conditions that severely reduce their capacity should flows suddenly be increased; a distinct possibility under controlled flow conditions where a sump pump may be turned on after being inoperative for some time. Problems of this sort can be mitigated somewhat by maximizing culvert diameter and flow velocity to the extent allowed by the performance requirements of the pond. In some cases this may be difficult, particularly for discharge culverts where it may be desirable to limit capacity to provide a higher detention time during storm events.

Problems can also arise when the ice in the vicinity of the inlet culvert discharge freezes over completely. In this situation the water may travel on top of the ice and drain to the discharge structure without any significant level of treatment. In order to avoid this, the operation of a pond must be monitored regularly so that if difficulties are encountered, holes can be broken in the ice in the immediate vicinity of the inlet culvert.

Ice loads on flow regulation structures, particularly drop inlet spillway culverts, can cause serious damage. During periods of low and/or intermittent flow, ice may come very close to, or actually become attached to, a drop inlet spillway. A subsequent change in water/ice elevation can then physically raise or lower the inlet to the point where irreversible damage results. To reduce the possibility of this kind of occurrence, discharge structures should either be designed structurally to tolerate ice loads or constructed in a configuration that minimizes the ice loads applied. An example of the latter is a principal spillway that consists simply of a straight culvert running longitudinally through the embankment with its upstream invert defining the pond high-water level during non-storm conditions. While the cost of such a culvert would be less than that of a drop inlet spillway, the embankment costs associated with it would be slightly greater since it would require a higher embankment to pass the same storm event with the same permanent pool elevation.

2.11.2.3 Inflow conditions. During the winter, groundwater flows pumped from disturbed area pits normally contribute the bulk of pond inflow volumes. This groundwater flow is usually considerably less than the design storm flow that the pond must be able to accommodate during the summer months. The relatively small volumes associated with groundwater flow often mitigate, to a large extent, the negative impact of low water temperatures and ice on winter settling pond performance.

2.11.2.4 Ambient air temperature. Low air temperatures can have an indirect influence on settling pond performance if some sort of coagulant aide is being used to enhance the flocculation process. These coagulant aides, be they polymers or metal salts, are typically injected into the pond's inlet channel from a small chemical mix/feed facility located onsite. If the building housing the equipment is inadequately or unreliably heated, the feed hardware may freeze on occasion, resulting in at least a temporary reduction in lagoon effluent quality. In addition, the chemical feed line discharging into the inlet channel is often susceptible to freezing if the chemical feed rate is low. In many cases these feed lines must be heat traced to avoid this problem.

2.11.2.5 Designing ponds for winter conditions. The impact of low water temperatures, ice, and reduced inflows on winter settling pond performance can be quantified using any of the physically based models described in Section 2.6. The relative merits of these models in this situation would be the same as those discussed previously for summer conditions. To evaluate pond performance under winter conditions, the following additional input parameters must be defined:

1. Minimum water temperature,
2. Maximum ice thickness, and
3. Maximum winter flows.

The minimum expected winter temperature could be conservatively assumed to be 0.5°C. The thickness of ice would vary slightly between areas, but could be estimated by tabulating thickness readings from other bodies of water in the local area. Maximum winter inflow could normally be assumed to be equal to the maximum groundwater flow estimated for the disturbed area pit in question.

Once the above parameters have been defined, pond performance can be evaluated using the methodologies described previously with the pond detention volume reduced to reflect the expected thickness of ice. If the expected groundwater flow is significant relative to the design storm flow, it would be prudent to evaluate a proposed pond for both summer and winter conditions. The pond configuration would then be modified as necessary to satisfy effluent quality requirements for the worse of the two operating conditions.

2.11.3 Overburden Characteristics

The wide range of overburden characteristics in the Alberta foothills and front ranges makes it very difficult to make general comments regarding the expected performance of settling ponds in this region. The fine-grained silts and clays that have the greatest impact on reservoir performance can be found in quantity in virtually any part of this area. Although the likelihood of encountering these materials is probably lower in this region than in the Alberta plains area, the probability of finding clay overburdens and/or partings at a particular site is still high enough to preclude the possibility of making any geographically based generalizations regarding expected settling pond performance.

The variable nature of overburden characteristics demands that meaningful estimates of sediment basin performance be based on site specific data. Those input parameters that are most strongly influenced by overburden characteristics are sediment yield and sediment particle size distribution. Methods for correlating overburden type to sediment yield were described previously in Section 2.5.1.2.1. Unfortunately, methods for predicting eroded particle size distributions from site-specific descriptions of overburden are not so clearly defined or widely available. The particle size distribution of the parent soil material provides only a poor estimate of the eroded particle size distribution. One of the few methods available for providing better size distribution estimates is described by Barfield, Moore, and Williams (1979, cited by Barfield, Warner, and Haan 1981). This method makes use of a rainfall simulator resembling the one shown in Figure 55. A soil sample is excavated from the watershed and placed in a pan with an expanded metal base. The sample is then subjected to a simulated rainfall that creates runoff from the sloped pan onto sieves nested in a 19 litre bucket. This simulated rainfall is continued until a depth corresponding to a design storm is reached. The sieves are then washed, removed, and the size distribution of fines determined from the runoff remaining in the bucket.

The relative merits of the various design methodologies discussed in previous sections are not strongly dependent on the characteristics of the overburden in a particular area. The most appropriate model for a given situation is the one that most

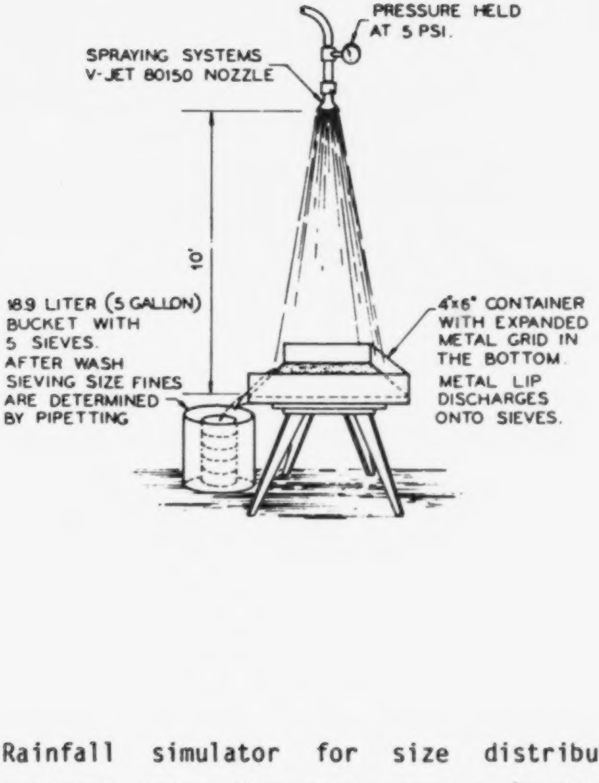

PRESSURE HELD
AT 5 PSI.
SPRAYING SYSTEMS
V-JET 80150 NOZZLE
10'
18.9 LITER (5 GALLON)
BUCKET WITH
5 SIEVES.
AFTER WASH
SIEVING SIZE FINES
ARE DETERMINED
BY PIPETTING
4"x6" CONTAINER
WITH EXPANDED
METAL GRID IN
THE BOTTOM.
METAL LIP
DISCHARGES
ONTO SIEVES.

Figure 55. Rainfall simulator for size distribution of eroded sediment using disturbed samples. (Barfield, Warner, and Haan 1981)

accurately evaluates a number of physical phenomena, not just those associated with overburden characteristics. Therefore, conclusions drawn regarding the suitability of a particular model or methodology would apply over a wide range of overburden and parting characteristics and it would normally not be necessary or advantageous to match models to watersheds based solely on the likelihood of encountering a particular soil type.

2.11.4 Wind

The Alberta eastern slopes often experience high winds, particularly in the spring and fall, which tend to increase the degree of short-circuiting and turbulence in a pond. Unfortunately, the literature search did not uncover any practically applicable design models that directly consider the effects of wind. Wind effects can be evaluated indirecly by using a single empirical factor in any of the physically based models described in Section 2.6; however, the literature did not provide any basis for the selection of this parameter as a function of wind speed and direction.

Although analytical procedures for quantifying wind effects are not well developed, their negative impact can be mitigated to some extent by incorporating certain features in the pond design. If space permits, ponds should be orientated so that their longitudinal axis is perpendicular to the prevailing wind direction. This reduces the pond fetch in the direction of wind and therefore reduces the maximum wave height for a given wind speed. Pond fetch can also be reduced by using baffles similar to those described in Section 3.1.10. Whatever mitigative measures are considered, their efficiency can be maximized if they are added during the planning stage, rather than after the pond is constructed.

3. ALTERNATIVES TO SEDIMENT REMOVAL BY PONDING

3.1 AUGMENTATIVE TECHNOLOGIES

3.1.1 Introduction

Augmentative technologies are used primarily to help control erosion and transportation of sediments. In some cases, they are also used to help increase the effectiveness of treatment systems (e.g., in-pond chemical treatment). Augmentative technologies are not alternate technologies, since none of these methods would be effective as a stand-alone technology. The technologies are used to minimize sediment erosion and transportation, thus reducing the sediment load to the treatment system. The technologies are generally used as part of a well-planned sediment control system.

Implementation of augmentative technologies requires a thorough understanding of up-to-date control practices and techniques, an understanding of limitations of the methods, and pre-planning to maximize the effectiveness of the techniques. The following sections discuss the various augmentative technologies, their applications, and their limitations.

3.1.2 Land Modification

There are various on-site and off-site surface modifications that can be used to reduce sediment loads on control systems during both the active and passive phases of a project. The specific actions taken will depend on the characteristics of the site, the nature of the land disturbing activity, and the overall concept for the sediment control scheme. Specific measures that can be taken include contour forming, scarification, pitting, land imprinting, and other surface alteration techniques. These can be useful preliminaries to other techniques discussed in subsequent sections and can be important elements in the control of sediment during the construction and reclamation phases of a project.

Contour forming is commonly used to reduce the amount of runoff and to enhance infiltration. It is normally practiced in arid climates and is most effective on gentle (less than 10%) slopes. Wischmeier and Smith (1978) have concluded that the maximum slope lengths for which contouring is effective range from 120 m on a 1% slope to 15 m on a 21% slope (i.e., the greater the slope, the shorter the effective slope length that can be treated).

To be effective, contour forming must alter or eliminate surface relief features that might encourage rill and gully erosion by stormwater runoff. A number of techniques are used to grade and shape soil surfaces. The main purpose is to roughen and loosen soil so that runoff is reduced and detained. Common methods used to modify land surfaces include (USEPA 1976):

1. Scarification,
2. Tracking,
3. Contour benching or furrowing, and
4. Pitting and land imprinting.

Scarification involves discing or harrowing along the contour. It can also be performed using a crawler tractor with grosser bars or by dragging the bucket teeth of a front-end loader over the ground.

Tracking is used on steep slopes unsuited to scarification. Shallow grooves parallel to the contour are generated by running a cleated crawler tractor up and down the slope.

Contour benching or terracing is done with a bulldozer cutting with a tilted blade so that small depressions and ditches are cut parallel to the contour. Furrowing is a similar procedure completed with a plough.

Pitting and land imprinting are techniques that have the same objective as benching and furrowing, but that use different equipment. Pitting or gouging can be accomplished using a specially designed tractor-drawn device that uses hydraulically operated disc scoops to create elongated basins on the contour. These basins are typically 0.35 m to 0.4 m wide, 0.15 m to 0.2 m deep, and 0.9 m to

1.2 m long. A variant of this technique is dozer basins, which are larger depressions (6 m long and 1 m deep) cut by a bulldozer operated with an inclined blade along the contour.

Simons et al. (1983) have proposed an impliment for land imprinting that resembles a sheepsfoot compactor. The heavy cylinder would be shaped to give a pattern of wedge-like depressions and short parallel troughs. They also recommend that imprinting can be practised on all soil textures on mild or steep slopes, but that pitting should be restricted to moderately dry, medium-textured soils with slopes less than 10%.

3.1.2.1 Effectiveness of land modification techniques. Data on the effectiveness of contour forming for sediment control are limited. The nature of the treatment is such that its effects are reduced in time by the establishment of new runoff channels and by revegetation, which will tend to fill in the grooves. These techniques are therefore best thought of as one of the first possible steps in a comprehensive program of sediment control that may include diversions and terraces as well as mulching, revegetation, or other more permanent control measures.

The Universal Soil Loss Equation (USLE) has been applied to predict the effectiveness of contour forming in the control of erosion. Stewart et al. (1975) recommend the use of the P factors (as defined in Section 2.5.1.2.1) shown in Table 45.

Some data on the effectiveness of pitting and land imprinting have been reported by Simons et al. (1983). In arid regions, pitting was observed to create depressions with a water storage capacity of 75 mm to 150 mm in the first year but only 2 mm in the second year. No quantitative data were found for the effectiveness of imprinting; however, it can be said that the practice would reduce erosion in small storms that do not significantly exceed the surface storage capacity. The effectiveness of pitting on erosion is given in Table 45.

Table 45. P factors for contouring.[a]

Land Slope (%)	Contouring[b]	Contour Furrows or Pits[c]
2 - 7	0.50	0.25
8 - 12	0.60	0.30
13 - 18	0.80	0.40
19 - 24	0.90	0.45
25 - 30	1.00	0.65

[a] Adapted from Stewart et al. (1975).

[b] Topsoil spreading, tillage, and seeding on the contour. Limitations on slope lengths 2% - 120 m, 8% - 60 m, 10% - 30 m, 14% to 30% - 18 m. Effectiveness beyond these lengths is speculative.

[c] Furrows or pits established on the contour.

3.1.3 Mulches

Mulch is a layer of plant residue or inorganic material applied to the soil surface to stabilize the soil, absorb rainfall energy, insulate against solar radiation, decrease evaporation, and aid in the establishment of plant growth. Mulching provides temporary protection against excessive soil erosion for periods of less than a year. It is often combined with a revegetation program.

The most common mulch materials are straw, wood chips, bark, excelsior, macerated paper, wood fibre, sawdust, and gravel. Table 46 gives the application rates of some of the more common mulch materials. Stabilization may also be provided by glass fibres, jute and plastic netting, and plastic filter sheet. These materials are not, strictly speaking, in the category of mulches but may be used in conjunction with more traditional materials. Chemical stabilization is used instead of mulching as a means of binding the soil particles to prevent wind and water erosion.

3.1.3.1 Effectiveness of mulches. The effectiveness of mulches in reducing soil erosion can be estimated from the C factor in the USLE (section 2.5.1.2.1). Very little site specific data on C factors for mulches are available. Simons et al. (1983) and Wischmeier and Smith (1978) have published ground cover ratios for straw, wood chips, and stones as a function of their application rates.

Generally, the effectiveness of mulches is limited by their resistance to the shearing forces exerted by runoff. Critical shear data are available only for cornstalks and straw (Foster 1982). Mulches are effective as a component of a well designed sediment control plan, but will not work as a stand alone technique.

The cost of mulching depend upon a number of site-specific factors including the type of mulch material, the area to be covered, the technique used to apply the mulch, the use of other materials (e.g. stabilization chemicals, seeds, fertilizers, lime), the distance to be travelled by machinery to reach the site, and the cost of labour. Comparative cost data for mulches show that the application

Table 46. Application of common mulch materials.

Material	Approximate Application Rate	Remarks
Straw	4-5 tonne/ha	Preferably anchored by a crimper or netting or chemical binder such as asphalt emulsion. Often used as a temporary cover. Inexpensive.
Wood Fibre	1.1-1.7 tonne/ha	Applied by hydroseeder. Favoured for steep slopes or flat areas.
Bark	50-60 m^3/ha	Sawmill waste. May be hogged; may lower soil pH.
Wood Chips	60 m^3/ha	Chipped waste from clearing operations. Only used on steep slopes if scarified or serrated. May lower soil pH. Machine or hand application.
Excelsior	-	Used in blanket form in drainageways and other critical areas to establish revegetation. Secured with metal staples. Expensive.
Paper	1.7 tonne/ha	Macerated newsprint; not widely used. Inferior to straw tacked with asphalt or wood fibre.

continued...

Table 46. Concluded.

Material	Approximate Application Rate	Remarks
Sawdust	500-1500 m^3/ha	50 mm to 150 mm blanket. Requires nitrogen fertilizer. Should be secured on steep slopes; subject to wind erosion.
Gravel, Stones	40-200 m^3/ha	Provides mechanical protection and allows vegetative growth. Suitable for low to moderately steep slopes.

of wood-fibre mulch using hydroseeders is the cheapest, followed by straw/hay, jute netting, wood chips, and excelsior mat in that order.

3.1.4 Vegetation

Revegetation of disturbed areas is one of the most important techniques that can be applied to prevent excessive soil erosion, both at active sites and in this permanent rehabilitation of an altered area. The effectiveness of the technique will depend upon the skill exercised in soil preparation, selection of plant species, and establishment of the plant community.

Grasses and legumes are the most commonly chosen species for control of erosion in the early stages of reclamation. In the longer term, trees and shrubs can provide a protective canopy and will contribute to the buildup of necessary surface organic materials by the deposit of leaf litter. Trees and shrubs are also important for purposes of screening, forestry, and wildlife protection.

Grasses and legumes well-suited to the site conditions can be chosen from a long list of possible species (see, for instance, tables IV-4, IV-5, and IV-6 in USEPA 1976). Properly selected grasses are highly adaptable to many site conditions and rapidly form a dense and lasting ground cover. Their root systems anchor the soil and allow surface water to infiltrate more rapidly. Their ability to spread themselves by surface and underground runners allows them to heal minor breaks in ground cover arising from erosion or other disturbances.

Legumes are generally used in combination with various grass species. Their major importance lies in their ability to fix atmospheric nitrogen, which is then available to non-nitrogen-fixing species such as grasses. They also have a large taproot, which stabilizes the soil and promotes infiltration. In mining, spoil areas are usually deficient in nitrogen and the presence of legumes will reduce the quantity of nitrogen fertilizer required to establish and maintain a grass cover.

In selecting grasses or legumes for a specific application, a number of criteria should be considered (USEPA 1976):

1. Ability to withstand the erosive and traffic stresses of the area being stabilized;
2. Adaptability to soil conditions (soil-water chemistry, moisture, texture, and fertility);
3. Adaptability to the climatic conditions of the site;
4. Resistance to insects and disease;
5. Adaptability to the post-activity land use;
6. Compatability with other plants selected for introduction or already present;
7. Ability to self-propagate; and
8. Maintenance requirements.

Trees and shrubs can be selected on a similar basis and are usually planted as seedlings.

A complete review of the literature specific to Alberta on the subject of revegetation has not been attempted. Much useful information on this topic is, however, available. Watson, Parker, and Polster (1980) have prepared a manual listing plant species suitable for reclamation in Alberta. Rothwell (1978) has described erosion control techniques associated with forest road-building. A bibliography of literature on revegetation of mining sites in northern Canada (Peterson and Peterson 1977) contains many references pertinent to Alberta.

3.1.4.1 Effectiveness of vegetation. The effectiveness of revegetation in preventing excessive soil erosion can be quantified by the cropping management factor C in the USLE. As noted in Section 2.5.1.2.1, this factor can be combined with the erosion control practice factor and equated to the product of three other factors. One of these is C_s, which accounts for surface stabilizing or protection treatments. Some values of C_s for seeded surfaces are given in Table 5. They vary from 0.40 for temporary seeding up to 60 days old, to 0.01 for permanent revegetation after 12 months. It is recommended that when plantings are followed by mulching, the lower C_s value should be used.

In an effort to further refine the C_s factor, Wischmeier (1975) recommended that it be considered the net result of three effects:

1. Type I: Canopy effect (vegetative canopies absorb the energy of the rainfall);
2. Type II: Mulch coverage (residue at soil surface reduces erosion); and
3. Type III: Root structure (root structure prevents erosion from overland flow).

The effective C_s factor then is defined as:

$$C_{eff} = C_I \times C_{II} \times C_{III} \tag{64}$$

Typical values of C_{eff} would be as follows:

1.	Bare soil	1.00
2.	Emerging vegetation	0.40
3.	Established grassland	0.01
4.	Established woodland	0.0001

The individual factors for C_I, C_{II}, and C_{III} can be estimated from graphs provided by Wischmeier (1975).

An examination of the C_s (C_{eff}) factors shows the importance of establishing vegetative cover to reduce runoff, increase infiltration, and reduce erosion. Therefore establishment of permanent vegetative cover should be the ultimate goal and a long-term solution to minimize sediment yield. It should be part of a well-designed sediment control plan. In the short term, while the vegetative cover is being established, it should be augmented by other off-site sediment controls to minimize downstream impact.

3.1.5 Diversion Structures

There are various diversion structures that may be used to intercept flow on sloping areas to reduce the volume of overland flow or to divert flow away from erosion sensitive areas. The outflow from these structures should be directed to a stabilized waterway. These techniques are not expected to be sufficiently effective by themselves to satisfy water quality standards for the effluent from an area under consideration. However, when used in conjunction with other measures, they can help alleviate serious problems. Long steep slopes can be terraced so that the downslope flow is interrupted periodically and diverted to a rip-rap waterway at one end of the slope. The overland flows are then reduced to levels that produce less soil erosion.

Diversion structures may be used to control sediment flow from activities such as surface mining by preventing runoff coming off exposed slopes from leaving the mine site without passing through sediment control facilities. They may also route runoff from higher undisturbed areas so that it does not add to the runoff from contaminated areas. They can also be used to protect lower portions of a hillside (spoil slopes and roadway grades) from erosion by surface flow by shortening the effective overland flow length.

This section discusses the following four types of structures:

1. Diversion dikes,
2. Diversion swales,
3. Diversion of drainageways, and
4. Grassed waterways.

A diversion dike is a ridge of compacted soil placed above, below, or around a disturbed area to intercept runoff and divert it to a disposal area. A diversion swale or ditch is an excavated, temporary drainageway used above and below disturbed areas to intercept runoff for the same purpose as a diversion dyke. A diversion is a permanent or temporary drainageway constructed by digging a ditch on a slope and building a soil dike on the downhill edge with the excavated soil. A grassed waterway is a natural or man-made drainageway of parabolic or trapezoidal cross-section that is below adjacent ground level and is stabilized by suitable vegetation. These definitions are taken from a summary of diversion design details (USEPA 1976) that was adapted mainly from standards and specifications developed by the U.S. Department of Agriculture. The most important of these recommendations are described in the following sections. Figures 56 through 60 give construction details for the various types of diversion structures.

Diversion dikes are considered to be a temporary measure to handle modest quantities of runoff. They may be used to prevent excessive erosion of a fill and cut slope until the slope is stabilized with vegetation or more permanent drainage features are provided. The general design criteria are as follows:

1. Drainage area of less than 2 ha;
2. Side slopes of 2:1 or flatter;
3. Sufficient grade to drain to an adequate outlet;
4. Stabilization of the channel area for slopes greater than 5%; and
5. Diverted runoff from a disturbed area conveyed to a sediment trap or an area protected by a similar device.

A diversion swale or ditch is used as a temporary measure to transport sediment-laden water to a sediment trapping device until the disturbed area is stabilized. It can also be used to prevent storm runoff from entering the disturbed area. General design criteria for a diversion swale are as follows:

1. Drainage area of less than 2 ha;
2. Level bottom with bottom width of at least 2 m;
3. Minimum depth of 0.3 m; and
4. Side slopes, grade, and stabilization as for diversion dikes.

The maximum spacing of swales is related to the slope of the right-of-way or disturbed area. The maximum distance recommended for a slope of greater than 10% is 30 m. For 5% to 10% slopes, and less than 5% slopes, the recommended values are 60 m and 90 m respectively.

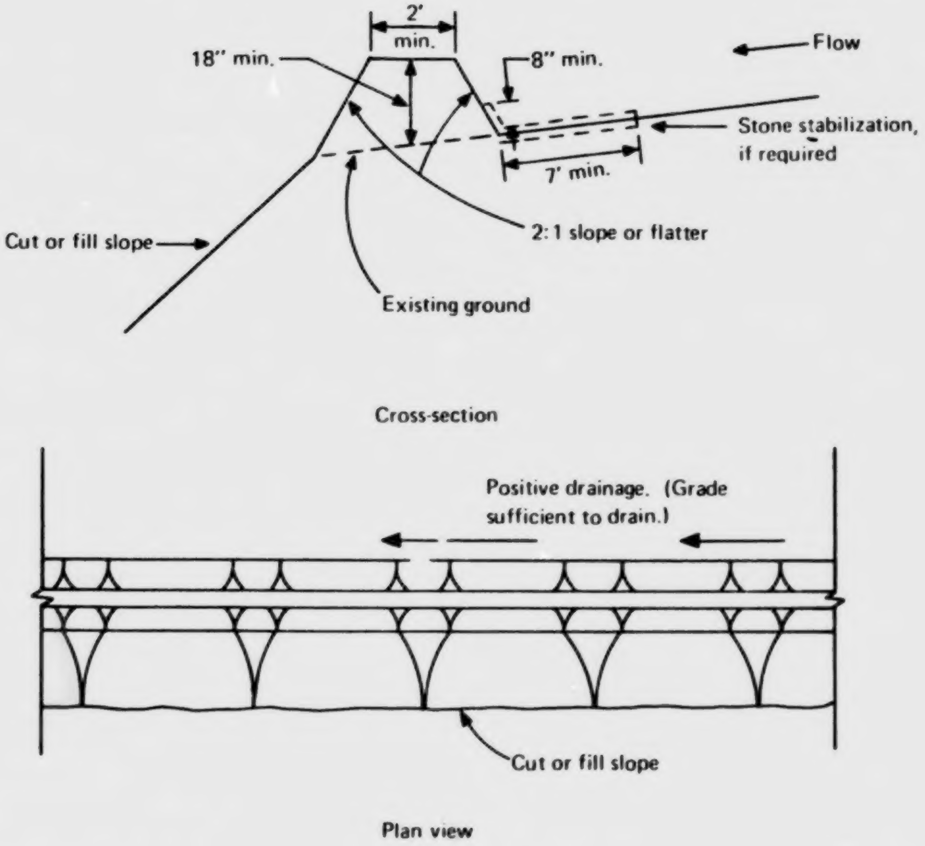

2' min.
18" min.
8" min.
Flow
Stone stabilization, if required
7' min.
2:1 slope or flatter
Cut or fill slope
Existing ground
Cross-section
Positive drainage. (Grade sufficient to drain.)
Cut or fill slope
Plan view

Figure 56. Diversion dike construction details. (USEPA 1976)

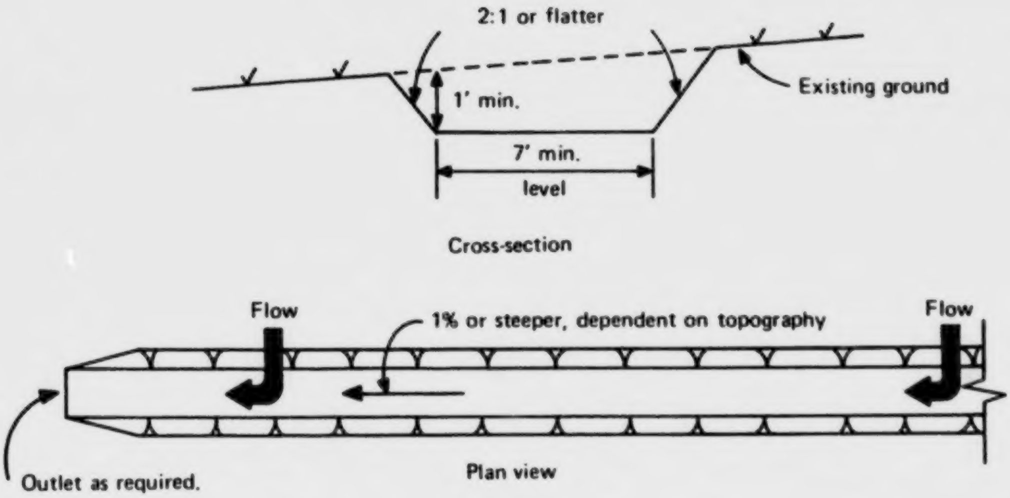
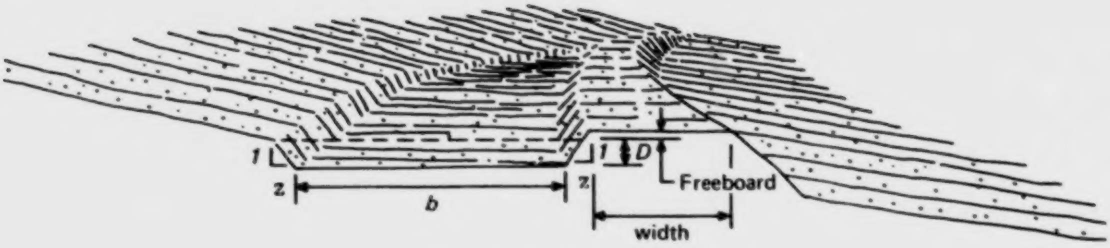
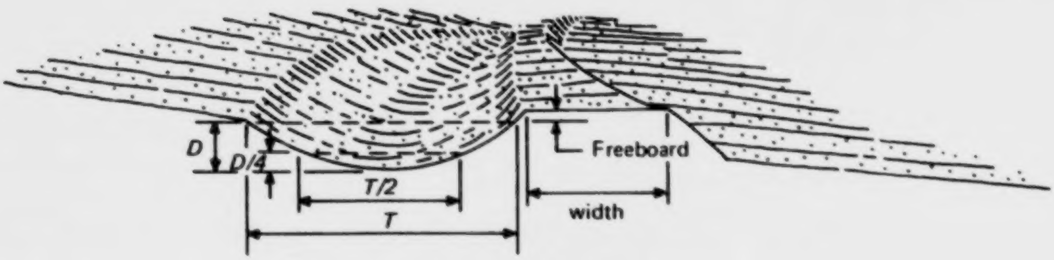

2:1 or flatter
1' min.
Existing ground
7' min.
level
Cross-section
Flow
1% or steeper, dependent on topography
Flow
Outlet as required.
Plan view

Figure 57. Diversion swale construction details. (USEPA 1976)


1
z
b
1
z
D
Freeboard
width

Trapezoidal cross-section


D
D/4
T/2
T
Freeboard
width

Parabolic cross-section

Figure 58. Diversion channel construction details. (USEPA 1976)

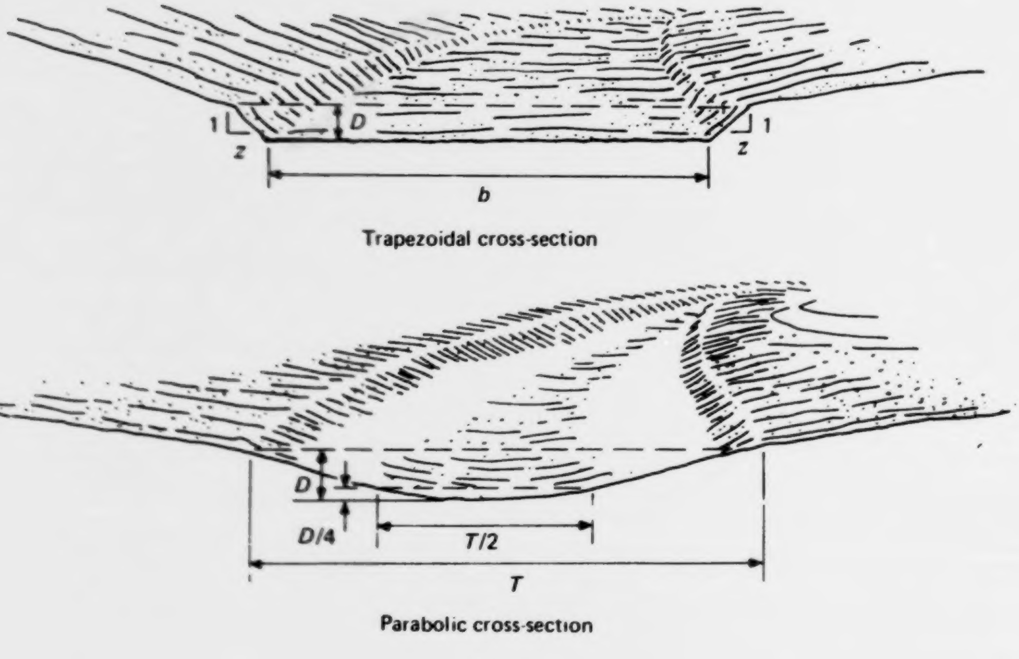
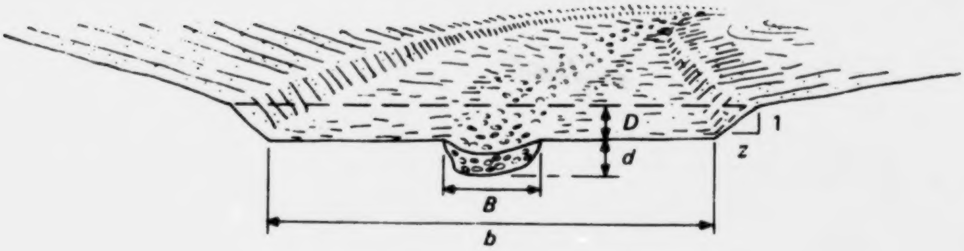
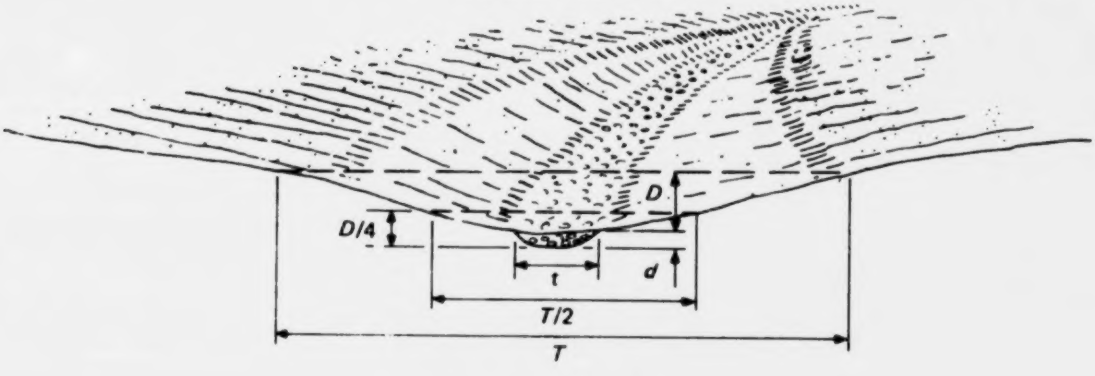

1
z
D
1
z
b
Trapezoidal cross-section
D
D/4
T/2
T
Parabolic cross-section

Figure 59. Grassed waterway construction details. (USEPA 1976)



Trapezoidal cross-section



Parabolic cross-section

Figure 60. Grassed waterway with stone centre construction details. (USEPA 1976)

Diversions of drainageways are primarily used to provide more permanent runoff control on long slopes subject to heavy flow concentrations. The design criteria include:

1. Diversion location should be determined after considering the layout of the site, topography, outlet locations, soil type, land use, length of slope, and possible seepage problems. The slope should not exceed 15%.
2. The capacity is determined by peak runoff values, as determined by standard engineering methods, and subject to the requirement of a minimum freeboard of 90 mm at the peak discharge of the design storm.
3. The cross-section is to be parabolic or trapezoidal, with side slopes no steeper than 2:1.
4. Stabilization as specified for a grassed waterway is required.
5. Maximum permissible design velocities are to be consistent with the vegetative cover and the maximum permissible grade.
6. The outlet must be stable (e.g., natural waterway or stabilized open channel), and the outlet must discharge in a manner that will not cause erosion.

A grassed waterway is a more permanent version of the swale and is used to convey runoff without causing damage by erosion. If there is a base flow, which may be the case with a natural drainageway, a stone centre, subsurface drain, or other suitable means should be specified to prevent sustained wetness and its adverse effect on the growth of vegetation. The major design criteria are as follows:

1. Minimum capacity is that required to confine the peak rate of runoff expected from the design storm (on slopes of 1% or less and if erosion or property damage is unlikely, this requirement may be waived).
2. Waterways are to be stabilized after excavation and compaction by seeding and mulching if the design

velocities are less than 1.0 m/s. At higher design velocities, jute or excelsior matting can be used to protect seeding, or the water can be temporarily diverted until the vegetation is established.

3. The outlet must be stable and discharge without causing erosion.
4. Design velocities are related to grade and the type of vegetative cover (a detailed design routine can be found in USEPA 1976).

Although it is not pointed out in the design specifications summarized above, it may also be necessary to consider the quantity of sediment carried into diversion channels and either allow volume for its deposition over the period of service or design the channel so that it will convey the sediment during the design storm. If the latter approach is adopted, measures must be taken to prevent erosion of the channel.

Costs of these engineering works have only two significant components, excavating and earth moving expenses and stabilization costs, and can be estimated by conventional methods.

3.1.5.1 Effectiveness of diversion structures. Diversion structures are effective if they divert water from erosion sensitive areas. The effectiveness of the measures employed depends on the number of structures utilized and the site conditions. Because of the site specific nature of diversion structures, very little information is available on their efficiencies. Chen (1974) reported that construction of three diversions on a slope reduced erosion by 50 percent. In general, if the measures intercept flow on sloping areas, reduce the volume of overload flow, or divert flow away from erosion-sensitive areas, then they can be said to be effective.

3.1.6 Check Dams

Check dams are structures used to stabilize the grade and control head in natural or artificial channels. They prevent excessive erosion by reducing flow velocities in watercourses and may contain partially lined sections or structures that can withstand high flow velocities. They may also function as sediment traps. In this section, three types of check dams are described:

1. Channel grade control devices,
2. Porous rock-filled check dams, and
3. Straw bale check dams.

Channel grade control devices are not intended to trap sediment, but to dissipate the potential energy of the flow in order to prevent erosion. The dam is an overfall structure made of concrete, metal, gabions,or wood. It is placed in a reasonably straight channel to give a step change in channel grade. The site must be selected to avoid erosion, sediment deposition, and flooding by peak storm runoffs. The drop structure is a similar installation consisting of a weir over which the water passes and drops to an approximately level apron or stilling basin. Detailed design specifications for drop structures can be found in USDA (1963).

Porous rock-filled dams are small end-dumped rock structures that are placed in drainageways to stabilize the channel and also to trap sediment. In some installations, filter fabric has been included in the design. An algorithm described by Barfield, Warner, and Haan (1981) and Hirschi and Warner (1983) predicts the efficiency with which sediment is removed by a porous rockfill dam. The model postulates quiescent settling in the backwater region upstream of the dam, a quiescent settling zone calculated from a single step back water curve, a flow through the porous dam determined from the porosity of the dam, and a triangular channel shape. The model assumes that trapping efficiency drops to zero if the dam overflows. The Hirschi model has been modified by Wilson, Barfield,and Moore (1982) and is included in the SEDIMOT II watershed model as a subroutine.

Check dams can be designed as sediment traps, in which case the dam height is selected so that the dam will not overtop under design flows. Hirschi's model, using either hand calculations or the

SEDIMOT II subroutine, is suitable for designing check dams as sediment traps.

If the design is for channel grade stabilization, Simons et al. (1983) recommend that the dam spacing be

$$S_d = H_d/0.55\ jS \tag{65}$$

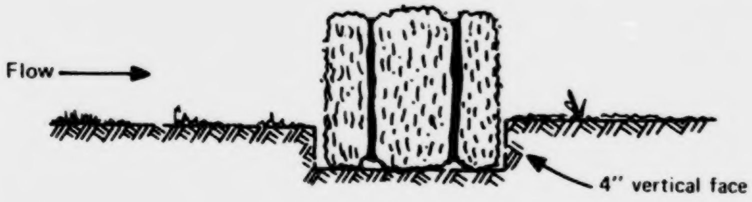
where S_d is spacing, H_d is dam height, and jS is the original channel slope.

Straw bale check dams are temporary structures built by cutting a 100 mm deep trench of the width of the bales, placing and staking the bales, and backfilling and compacting the excavated soil against the upstream and downstream sides of the bales. They are temporary structures with a life expectancy of about three months and can be installed across or at the toe of a slope. Their purpose is to trap small amounts of sediment from unprotected areas of less than 0.2 ha. They can also be used as temporary barriers at storm drain inlets, drainageways, and across right-of-ways. Construction details of a typical dam are shown in Figure 61.

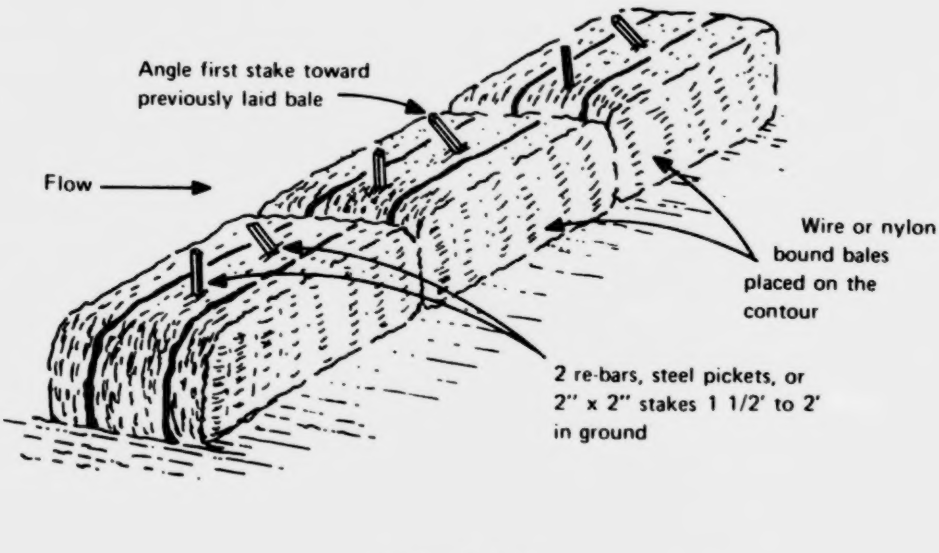
Straw bale check dams are normally one bale high and the only design consideration of importance is the determination of the length of the dam. If a channel is being specified, its width should be such that the straw bale dam is not overtopped during a design storm.

3.1.6.1 Effectiveness of check dams. Check dams are effective components of a well-designed sediment control plan, but they will not meet an effluent total suspended solids or settleable solids standard. They are often used to reduce the sediment deposition in settling ponds. Trapping efficiencies of porous check dams are generally low and range from near zero for fine sediments to 25 to 30 percent for coarse materials.

The effectiveness of straw bale check dams in trapping sediments has been analyzed in the same manner as porous rockfill


Flow
4" vertical face

Embedding detail


Angle first stake toward previously laid bale
Flow
Wire or nylon bound bales placed on the contour
2 re-bars, steel pickets, or 2" x 2" stakes 1 1/2' to 2' in ground

Anchoring detail

Figure 61. Straw bale check dam construction details. (USEPA 1976)

check dams by Barfield, Warner and Haan (1981). The flow through velocity for the bales was taken to be 0.011 m/s rather than being determined by its porosity. The results show that the straw bale check dams have a higher trap efficiency than the porous rockfill check dam. Typically though, a straw bale check dam will not meet a total suspended solids or settleable solids standard over a long period of time. The bales tend to seal over time due to trapped sediment and fail to work as a filter. Practical experience with straw bale check dams has not been positive for many practicing engineers. This is thought to be largely due to poor installation practices.

3.1.7 Filter Fences

A filter fence consists of a conventional fence supporting a porous fabric, either synthetic or natural, sealed at the bottom by compacted backfill. It is primarily used to remove sediment from runoff draining from a disturbed area. The fabrics used are of low permeability and thus can only be used for overland flows and not for channelled flows. A filter fence is a temporary measure used to control sediment transport. To maintain its effectiveness, trapped sediment should be removed when it reaches 50% of the height of the fence.

In principle, the sediment filtering model used for straw bale check dams can be used to predict the effectiveness of filter fences. The flow-through velocity needed in this calculation depends on the fabric used. In practice, the usefulness of such calculations, in view of the uncertainties in the data required, is questionable.

3.1.7.1 Effectiveness of filter fences. Filter fences should be more efficient in trapping finer particles than check dams. Their efficiency depends on the installation and the integrity of the fabric. Breaks can be caused by vandalism and wildlife, and necessitate frequent inspection and repair to render the fence effective.

3.1.8 Vegetative Filter Strips

Vegetative filter strips consist of a temporary or permanent area of vegetative cover positioned to intercept storm water before it enters a watercourse or treatment facility. As water flows across the strip, a portion of the suspended solids is removed by the vegetation and deposited. The flow velocity is also reduced, resulting in increased infiltration and sediment removal.

The vegetative strip can be natural or man-made, and a tall, dense stand of grass is considered to be most suitable. Naturally occurring vegetation, if present, may be enhanced by fertilization. Vegetative strips can also be established by laying sod or by seeding.

The effectiveness of vegetative filter strips in removing sediment has been studied in considerable detail in recent years. A series of experimental and theoretical studies have shown that vegetative strips can be very effective in removing sediment. This finding has resulted in a simulation model known as GRASFIL, which is a subroutine of the SEDIMOT II computer program (Tollner et al. 1976, 1977; Hayes, Barfield and Barnhisel 1979a, 1979b, 1982, 1984; Barfield, Hayes, and Barnhisel 1978; Barfield and Hayes 1980).

The GRASFIL model describes the vegetative filter strip in terms of four zones, variable in time, which are characterized as follows:

1. Zone A: vegetation is completely covered with sediment and there is no deposition (i.e., total transport), A(t);
2. Zone B: deposition wedge is formed characterized by uniform deposition with downstream distance, B(t);
3. Zone C: sufficient deposition on the bed so that bed load transport occurs but the channel slope does not change, C(t); and
4. Zone D: insufficient deposition on the bed, all sediment reaching bed is trapped, D(t).

In each zone, sediment capture is modelled by appropriate mechanistic arguments and the development of the deposition is obtained from a sediment balance over the entire bed. The model is shown conceptually in Figure 62.

Trapping efficiency for suspended load is calculated by an empirical equation in which the probability of being trapped is assumed to be proportional to the number of times a particle settles to the bed in its trip through zone D(t), and inversely proportional to the turbulence level as modeled by the Reynold's number. At the inlet section it is assumed that the sediment forms a horizontal profile directly upstream of the filter.

Inputs to the GRASFIL subroutine are:

1. Filter slope,
2. Filter width,
3. Media spacing,
4. Stiffness factor of vegetation (used to determine if the vegetation bends over as flow increases),
5. Manning's n,
6. Infiltration rate (steady state),
7. Media height,
8. Number of slope segments, and
9. Bulk specific gravity of deposited sediment.

GRASFIL output parameters include:

1. Hydrograph (effluent),
2. Sedimentgraph of TSS (effluent),
3. Sediment particle size distribution, and
4. Settleable solids (average and peak).

The accuracy of the GRASFIL model has been verified by field and laboratory tests (Hayes, Barfield, and Barnhisel 1984; Albrecht and Barfield 1981). Hayes, Barfield, and Barnhisel (1984) describe the results of field tests where data were collected on five filters with slopes ranging from 2.8% to 20% with a total length of nearly 30 m. Fescue was the chosen vegetation in all cases. An example of predictions of effluent TSS concentrations is shown in

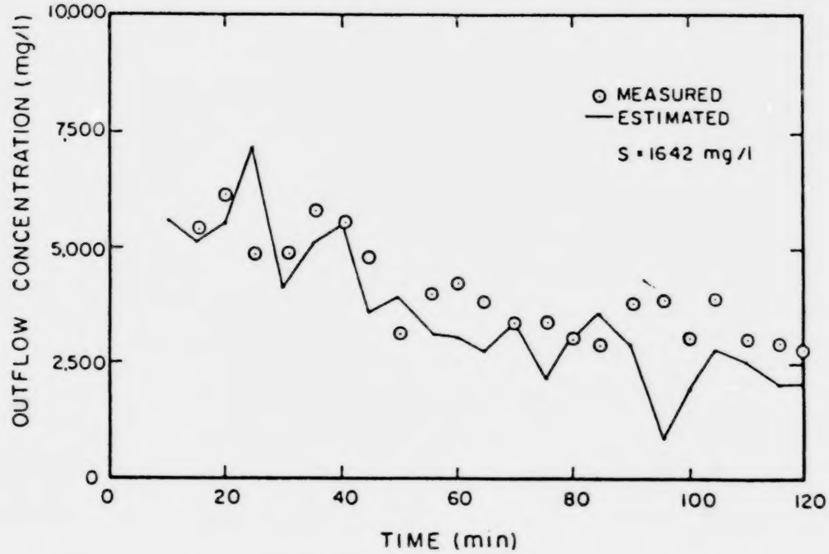


Figure 62. Example of outflow concentrations predicted by the GRASFIL model. (Hayes, Barfield, and Barnhisel 1982)

Figure 63. The agreement between observed and predicted concentrations was excellent. Albrecht and Barfield (1981) also found that the model adequately predicted trapping efficiency for a large-scale field model.

The design of the vegetative filter is based on trial runs with the GRASFIL model. The filter must be adequate to accommodate flow without vegetation bending over or filter submergence. Thus, both grass height and filter width are important design factors. Other practical considerations include:

1. Provision of a level spreader at the upper edge of the filter to prevent channelization of flow;
2. Uniform slope from side to side;
3. Regular mowing of the filter during its growing period; and
4. Provision for diversion of surface flow during the period of establishment of vegetation.

3.1.8.1 Effectiveness of vegetative filter strips. Wilson (1967) showed that vegetation could filter sediment from flow. Since then, laboratory and field trials have demonstrated that vegetative filters have a very high sediment trapping efficiency in the order of 60 to 90 percent. A distinct advantage of a vegetative filter is that it requires minimal maintenance compared to other augmentative sediment controls, hence it has the potential for long-term application.

3.1.9 Sediment Traps

Sediment traps are small temporary basins formed by excavations and/or embankments. They intercept runoff and trap and retain the coarser fractions of the sediment. They are usually installed in roadway ditches or drainage ditches or other points of discharge from a disturbed area. They differ from check dams in that they are not intended to control grade or velocity. Sediment traps do not have special inlet controls, and in their simplest form are merely an

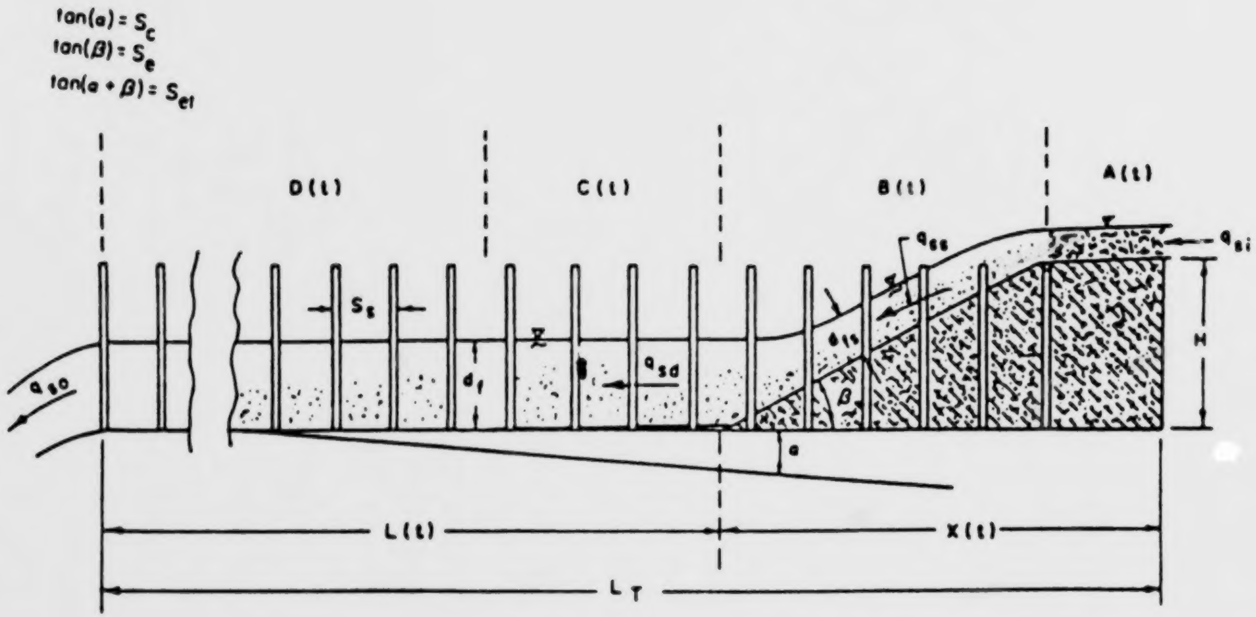

tan(α) = S_c
tan(β) = S_e
tan(α + β) = S_el
D(t)
C(t)
B(t)
A(t)
q_si
q_ss
S_s
H
d_f
q_sd
β
α
q_so
L(t)
X(t)
L_T

Figure 63. Schematic of model sediment trapping in a sediment filter. (Barfield and Hayes 1980)

excavation in a ditch. There are variants that have an outlet pipe and riser, or a crushed stone facing on the overflow embankment.

Detailed construction specifications of sediment traps are described in USDA (1975) and USEPA (1976). Two examples of traps are shown in Figure 64. The major points in the design of an earth outlet sediment trap are described in Table 47.

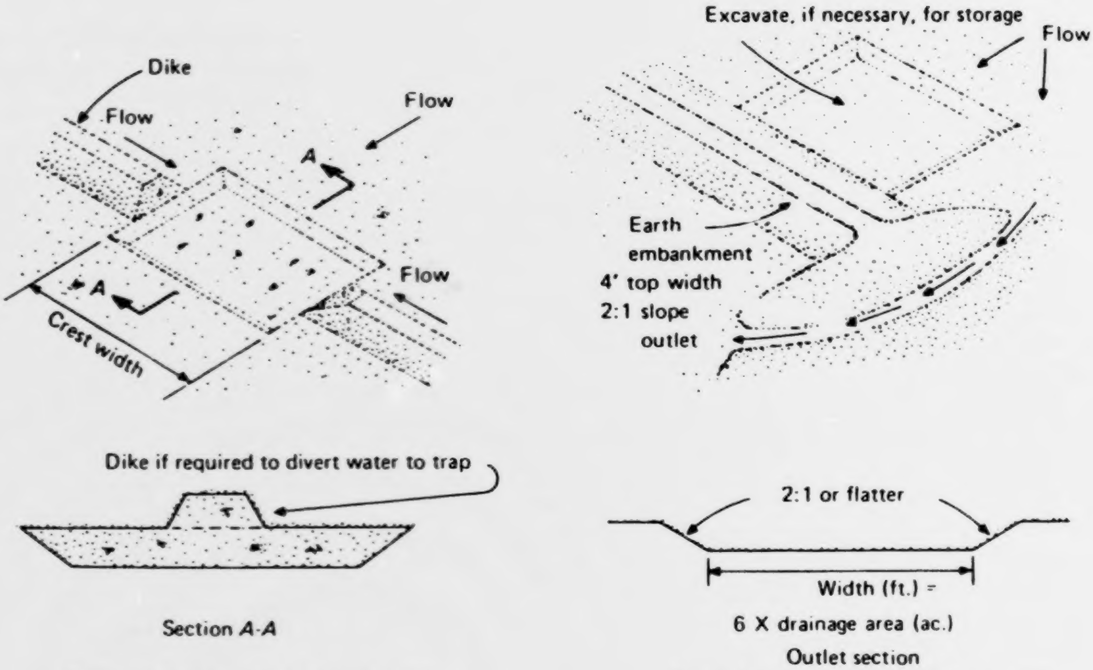
The pipe outlet sediment trap consists of a basin with a perforated riser and a drainage pipe passing through an embankment. Recommended pipe diameters range from 300 mm for a 0.4 ha drainage area to 760 mm for 2.0 ha. The stone outlet sediment trap has an outlet that allows flow over a crushed stone section on an embankment. The embankment consists of a relatively impervious core (e.g., timber, concrete blocks, or straw bales) with a cover of 150 mm of crushed stone. Construction specifications can be found in the two references cited above.

In the case of the earth outlet sediment trap, outlet protection can be provided by natural vegetation, grass, rock rip-rap, or fibre mats.

3.1.9.1 Effectiveness of sediment traps. The effectiveness of a sediment trap depends on the size, flow rate, and incoming sediment size distribution. In this respect, the trap acts like a miniature pond. A detailed estimate can be made using methods described previously in Section 2.

3.1.10 Settling Pond Modifications

The performance of a settling pond is adversely affected by deviations from the idealized concept of plug flow (e.g., turbulence and resuspension of solids). Deviations from plug flow cause short-circuiting and dead storage, both of which represent an inefficient use of the volume of the settling pond. Resuspension of solids is the consequence of local excessive velocities. All these adverse effects become worse as the influent flow increases. Mitigative measures can be taken to minimize these effects in existing sedimentation ponds.


Dike
Flow
Flow
A
Flow
A
Crest width
Dike if required to divert water to trap
Section A-A
Excavate, if necessary, for storage
Flow
Earth embankment
4' top width
2:1 slope
outlet
2:1 or flatter
Width (ft.) =
6 X drainage area (ac.)
Outlet section

Excavated earth outlet sediment trap

Embankment earth outlet sediment trap

Figure 64. Earth outlet sediment traps. (USEPA 1976)

Table 47. Sediment traps design specifications.

Drainage area:	Less than 2.0 ha
Trap Size:	125 m^3 per ha of drainage area
Embankments:	Not to exceed 1.5 m in height as measured at the low point of the original ground along the centre line of the embankment; 2:1 or flatter side slopes, minimum 1.2 m wide top; compaction only from equipment traversing during construction
Excavations:	Side slopes of 2:1 or flatter
Trap Cleanout:	When sediment has accumulated to half the design depth it should be removed

A well-designed settling pond should incorporate the following features (Ward, Haan and Tapp 1979):

1. Inlet to outlet distance greater than twice the width;
2. A wedge shape with the inlet at the narrow end;
3. A minimum average pool depth of 0.9 metres;
4. A minimum length between inlet and outlet of 2.25 m for each hectare; and
5. A surface withdrawal system.

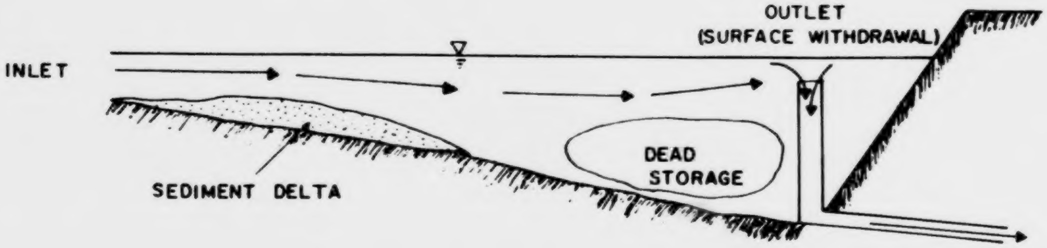
Many existing ponds do not wholly satisfy these criteria and the quality of their discharges may be improved by various modifications designed to improve the circulation patterns in the pond. These measures can be conveniently classified as:

1. Changes to the inlet flow;
2. Changes to the spillway or withdrawal system;
3. Diversion of circulation in the pond;
4. Use of aquatic vegetation in the pond.

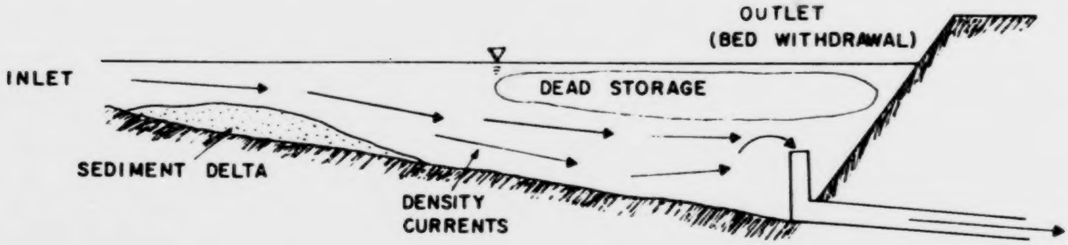
Ward, Williams and Haan (1979) describe the consequences of a pond which does not satisfy the criteria of a minimum length to width ratio of two. The inlet flow, if it contains high concentrations of sediment, will form a density current which flows along the bed of the pond. If the pond is short, and if the inlet velocity is relatively high, the flow will be influenced by the outlet and will bypass most of the pond volume. If the outlet flow is withdrawn close to the surface, a dead storage zone will be created near the bottom. Conversely, withdrawal near the bottom will create a dead zone near the surface (Figure 65).

Ward, Williams and Haan (1979) point out that the 2:1 length to width criteria is not sufficient to prevent short circuiting unless the peak flow rates and the permanent pool volume are in the correct relationship. If both conditions are satisfied, the pond performance is not controlled by short circuiting and there is little advantage in increasing the length-to-width ratio. They also suggest that the flow pattern most desired is one in which the density current stays near the bottom for most of its passage down the length of the

Short-circuiting with a surface withdrawal outlet


OUTLET
(SURFACE WITHDRAWAL)
INLET
DEAD
STORAGE
SEDIMENT DELTA

Short-circuiting with a bed withdrawal outlet


OUTLET
(BED WITHDRAWAL)
INLET
DEAD STORAGE
SEDIMENT DELTA
DENSITY
CURRENTS

Circulation in a well-designed sedimentation pond

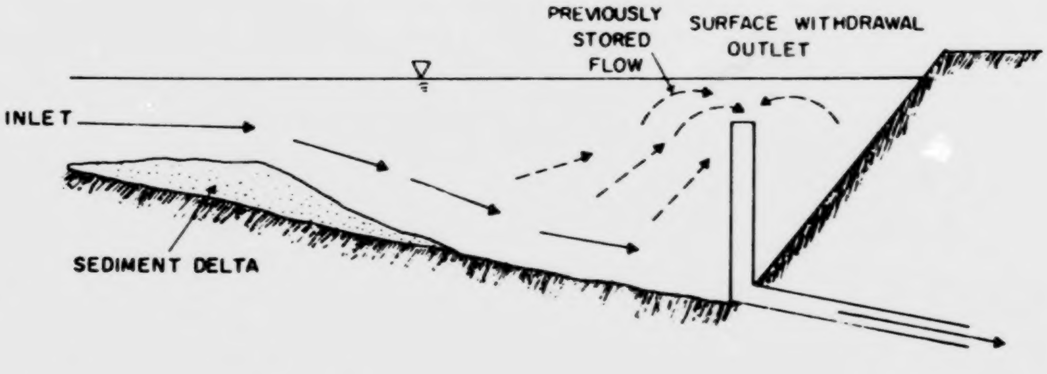

PREVIOUSLY
STORED
FLOW
SURFACE WITHDRAWAL
OUTLET
INLET
SEDIMENT DELTA

Figure 65. Circulation in sediment control ponds. (Adapted from Ward, Haan, and Tapp 1979)

pond and the upper layers of the current are drawn upwards to an outlet located near the surface (Figure 65). Modifications that encourage this flow pattern should be considered if a settling pond is performing poorly.

Short-circuiting is also more likely when the inlet flow possesses sufficient kinetic energy to create a turbulent jet. This type of flow will entrain surrounding fluid and encourages a recirculating pattern as shown in Figure 66.

From the above discussions it may be concluded that any changes to the inlet flow that reduce its kinetic energy and distribute it as widely as possible across the inlet end of the pond are most desirable. Two measures of this nature have been described by Ward, Haan and Tapp (1979). One is the inlet overflow baffle, which is a low weir structure that extends across the width of the pond (Figure 67). This modification will distribute the inlet flow effectively but unless provision is made for regular cleaning of sediment from the upstream side of the weir, it may become ineffective as the forebay area becomes filled with sediment.

The second measure suggested by Ward, Haan,and Tapp (1979) is the inlet directional baffle, which is simpler and needs less operating attention. It consists of a rough lumber or log structure (Figure 68) that is placed near the inlet to deflect the inlet stream, either into two flows or towards one side of the pond (Figure 67).

Baffles may also be placed more centrally in the pond. Ward, Haan, and Tapp (1979) describe a central baffle to break up short-circuiting and oblique baffles to compensate for assymetrical placement of the inlet with respect to the outlet (Figure 69).

Jenkins and Pankanin (1979) describe the improvements in effluent quality from two sediment ponds at a surface coal mine in West Virginia that were achieved by adding a central baffle and modifying the outlet. The length-to-width ratio of the pond was 1.7. The length per hectare of drainage area was 6.3 metres, which is considerably greater than the minimum 2.3 metres recommended by Ward,

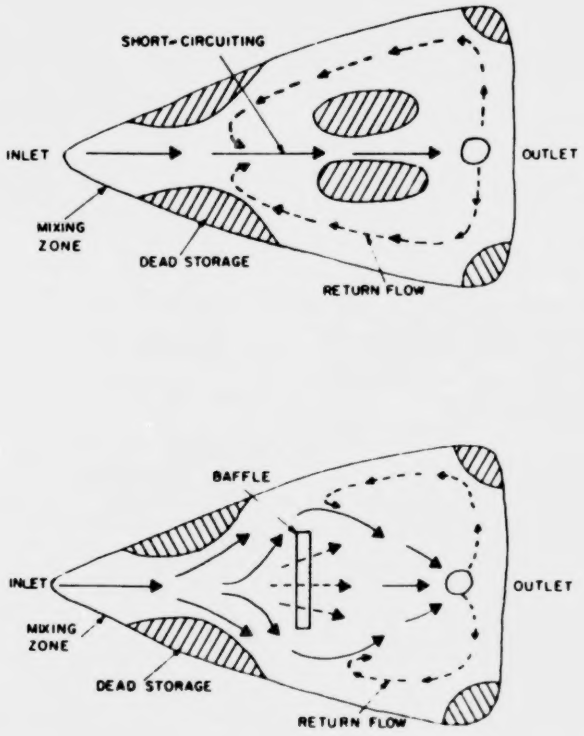
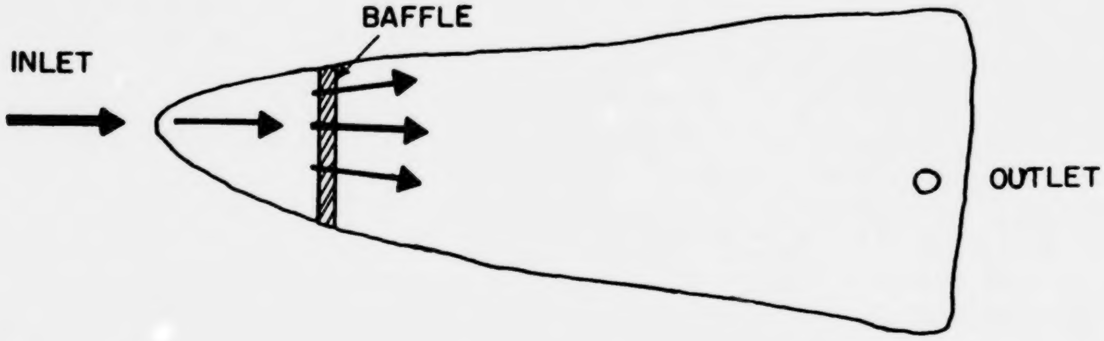
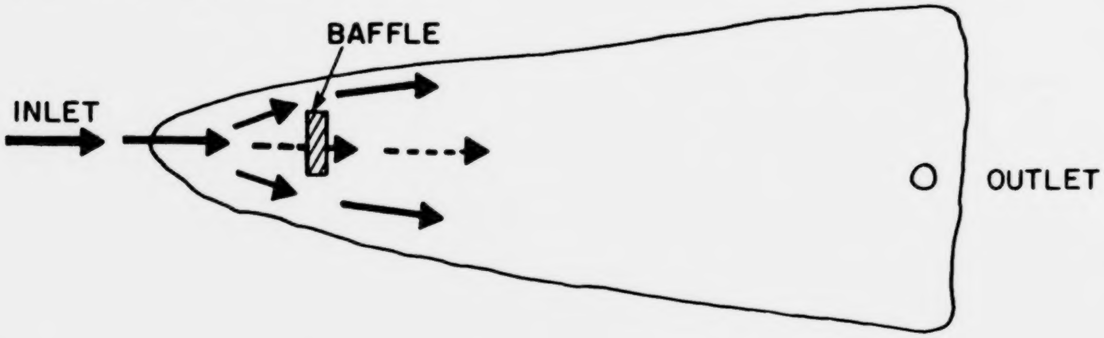

SHORT-CIRCUITING
INLET
OUTLET
MIXING ZONE
DEAD STORAGE
RETURN FLOW
BAFFLE
INLET
OUTLET
MIXING ZONE
DEAD STORAGE
RETURN FLOW

Figure 66. Sedimentation pond flow patterns. (Ward, Haan, and Tapp 1979)

Inlet overflow baffle


BAFFLE
INLET
OUTLET

Inlet directional baffle


BAFFLE
INLET
OUTLET

Inlet directional baffle

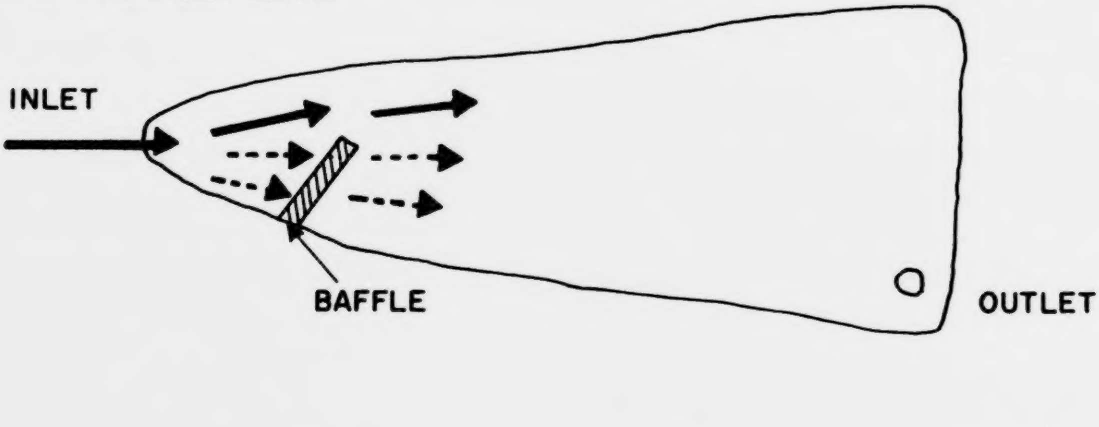

INLET
BAFFLE
OUTLET

Figure 67. Inlet baffle systems.(Ward, Haan, and Tapp 1979)

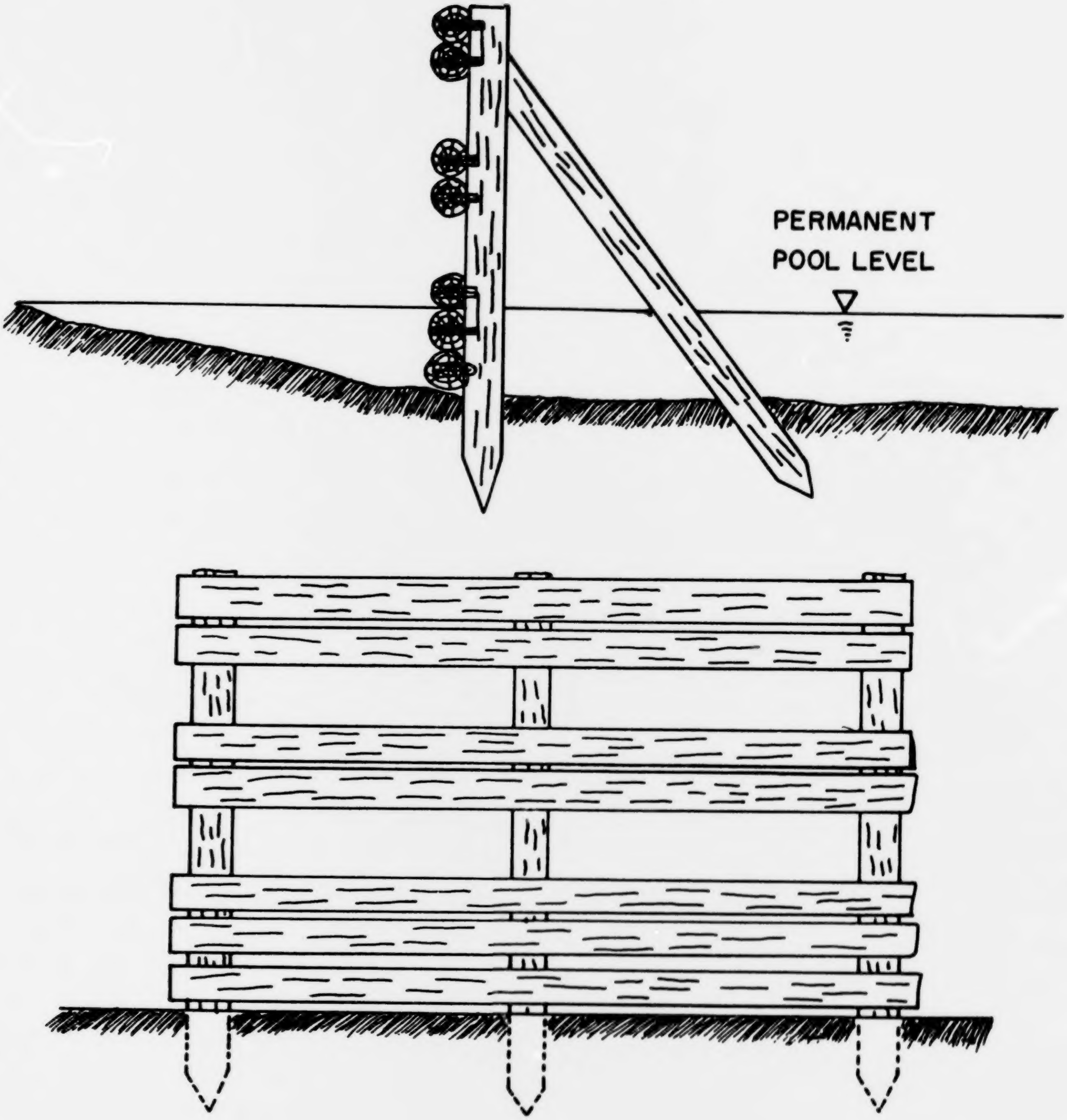
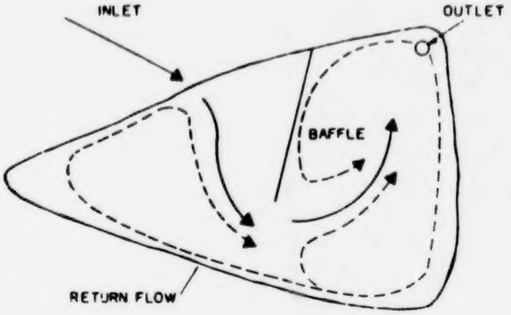

PERMANENT
POOL LEVEL

Figure 68. Typical inlet baffle design.(Ward, Haan, and Tapp 1979)


INLET
OUTLET
BAFFLE
RETURN FLOW

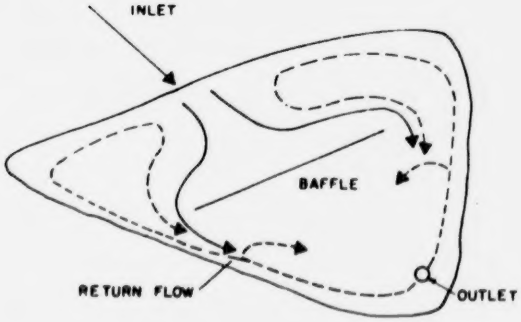

INLET
BAFFLE
RETURN FLOW
OUTLET

Figure 69. Pond directional baffle systems. (Ward, Haan, and Tapp 1979)

Haan, and Tapp (1979). The central baffle consisted of a 1.4 m x 22.9 m sheet of brattice cloth supported by floating logs connected by nylon rope. The baffle spanned the pond and was weighted down at the lower corners. The average pond depth was 2.0 m, so that the effect of the baffle was to divert the inlet flow under it. Prior to this modification and an improved outlet arrangement (to be discussed later), effluent solids concentrations often exceeded the maximum legal limit of 70 mg/L. No violations were observed after the changes.

Taylor and Jenkins (1982) report trials on four ponds with similar baffles. The first pond conformed to the good design practices recommended by Ward, Haan, and Tapp (1979), but performance was significantly improved by addition of the central baffle. The second pond was in poor condition, had been baffled prior to the study, and was almost full of sediment. The third pond satisfied the criteria for good design except that its length-to-width ratio was 1.8. The change in performance due to the installation of the baffle was not established in the trials because, during the relatively short trial duration, influent conditions with and without a baffle were not comparable. The fourth pond was rectangular, with inlets on two adjacent sides and a floating outlet in the opposite corner. A baffle placed obliquely in front of the outlet, about half the bank-to-bank distance in length, was used to correct short-circuiting. Total suspended solids concentration in the effluent was significantly reduced by the baffle.

Recommendations have been made for improvements to outlet pipe design. A common design makes use of a horizontal pipe with a perforated riser. A disadvantage of this device is that the largest flow is through the lowest perforations. Various suggestions have been made for improvements (USDA 1975, USEPA 1976), but there are few reports of field applications. Ward, Haan, and Tapp (1979) have recommended taking advantage of the wedge shape of the sedimentation pond by using twin outlets (Figure 70), but no trials appear to have been done.

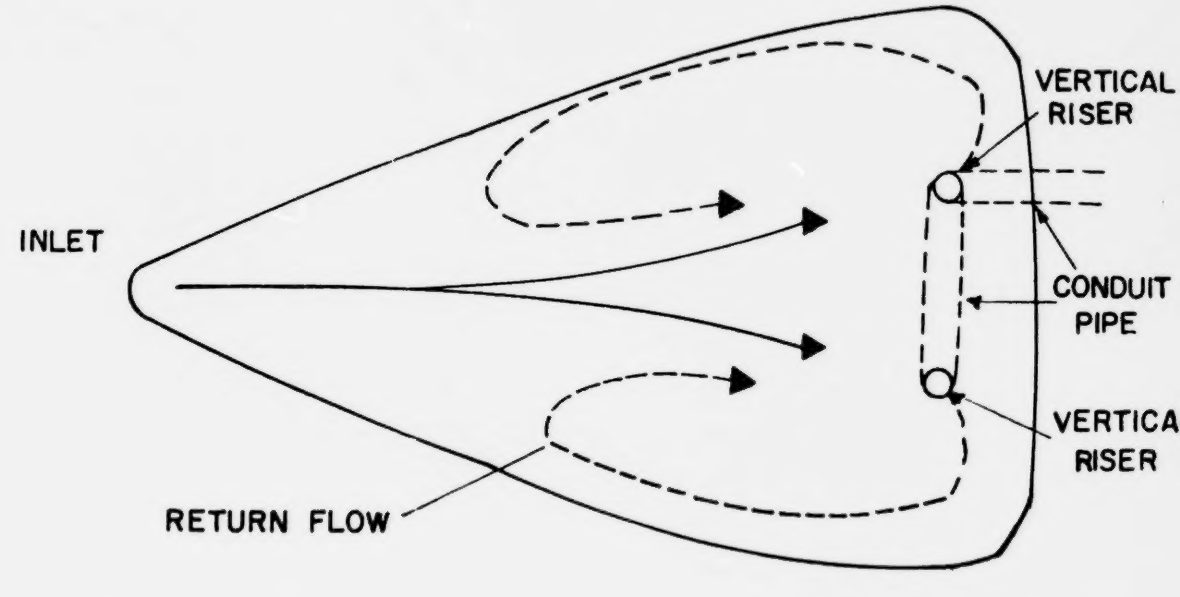

VERTICAL RISER
INLET
CONDUIT PIPE
VERTICA RISER
RETURN FLOW

Figure 70. Probable flow patterns with a twin riser spillway. (Ward, Haan, and Tapp 1979)

Outlet pipe design was modified in the mini-water sediment control ponds studied by Jenkins and Pankanin (1979) and Taylor and Jenkins (1982). Jenkins and Pankanin replaced a 500 mm diameter horizontal pipe used for an outlet by a weir box 7.3 m long with vee-notches on its front side. Variations in pond level caused some difficulties with this structure. The authors did not measure the influence of the central baffle separately from that of the modified outlet on the improved performance. In a second pond, the only modification was to the outlet, which was originally a 915 mm diameter riser with 40 mm diameter holes spaced evenly over the top 600 mm. A 3.05 m x 3.05 m x 0.90 m wooden frame covered on the outside by fibreglass sheets was placed around the riser, forming a calming well. It was submerged 0.3 m and was intended to prevent short-circuiting across the pond surface. This modification was not successful, partly because during the test the clearance between the sediment layer and the bottom of the frame was so small that significant quantities of solids were resuspended.

Taylor and Jenkins (1982) modified the ponds described above using a siphon-type outlet and a floating outlet,which are shown in figures 71 and 72. Both are designed to remove water from as close to the surface as possible. The siphon-type outlet gave better efficiency results when the central baffle was used, but the data collected do not show any improvement in pond performance over the pond having only baffles. There is insufficient performance data on the effect of the floating outlet at the time of reporting for any conclusions to be reached.

The small siphon (Figure 71) was designed for a flow slightly less than the flow through the lowest row of perforations on the riser. It handled most flows except during heavy runoff, when the large siphons came into play. The larger siphons did not prime until the level had risen by 0.46 m,and it was found that it took about two days for the siphons to draw the pond down to the level at which the suction was broken on the large siphon.

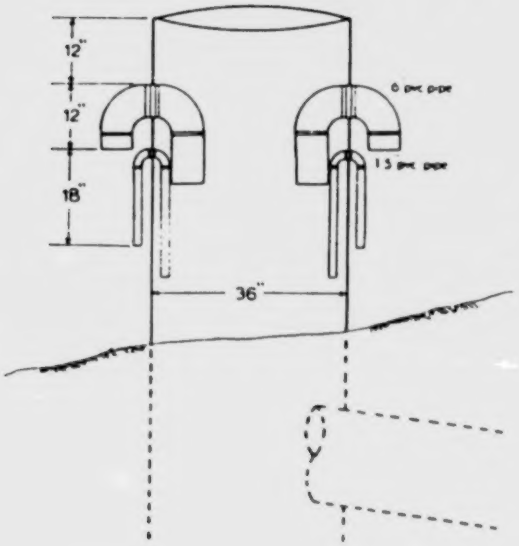
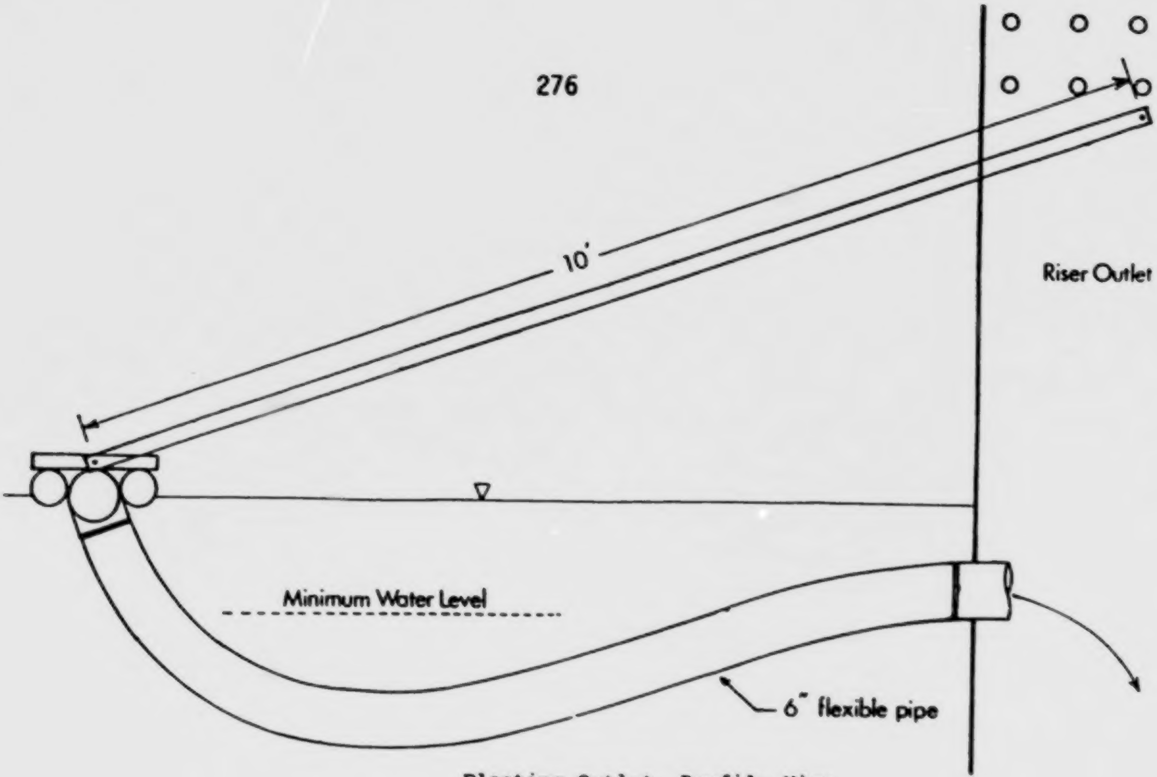
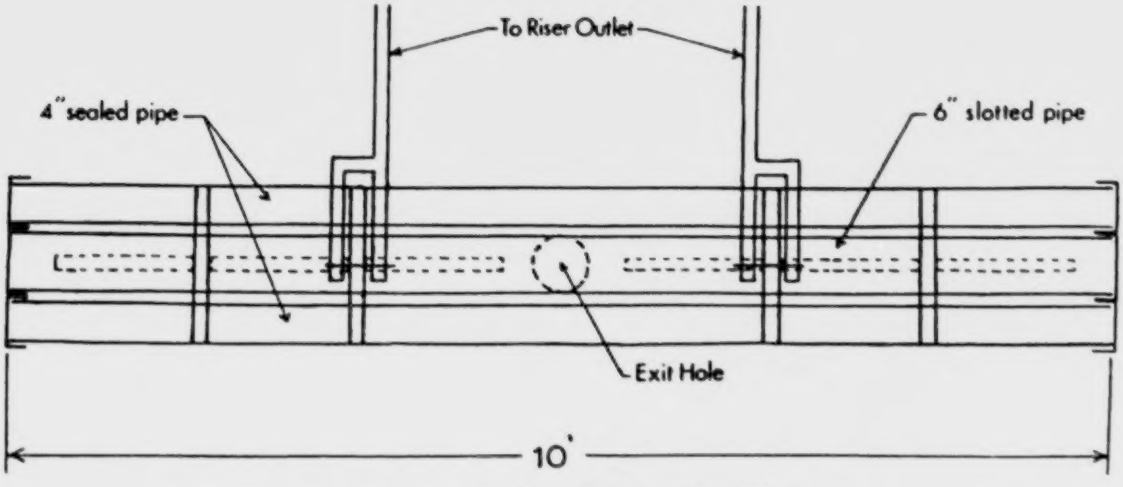

12"
12"
18"
6 pvc pipe
1.5 pvc pipe
36"

Figure 71. Typical siphon outlet.(Taylor and Jenkins 1982)


10'
Riser Outlet
Minimum Water Level
6" flexible pipe

Floating Outlet, Profile View


To Riser Outlet
4" sealed pipe
6" slotted pipe
Exit Hole
10'

Floating Outlet, Plan View

Figure 72. Typical floating outlet. (Taylor and Jenkins 1982)

3.1.10.1 Effectiveness of settling pond modifications. The above studies show that improvements in the performance of sediment control ponds can be obtained by inexpensive modifications. The costs of baffles and siphon outlets are minimal compared to other forms of treatment or modifications.

3.1.11 In-Pond Chemical Treatment

In-pond chemical treatment is used to enhance removal of finer particles which do not settle by themselves. In its basic form, the system makes use of chemicals fed directly from containers. The pond is used both as a mixing basin and as a settling basin. The objective is to promote coagulation and flocculation, thus enhancing settling. In a more controlled operation, the chemicals which may be a polymer or an inorganic metal salt, are diluted and fed at predetermined concentrations at a dosage which is set proportional to the flow. Mixing is provided either in-line or in a separate mixing zone to promote dispersion of chemicals and to enhance flocculation. Depending on the properties of the chemicals used it may be possible to feed the chemicals neat, thus avoiding importation of fresh water to dilute the chemicals.

In-pond chemical treatment can be viewed as a technique of extending settling pond technology to remove fine particles. In this respect, experience shows that the treatment can be cost-effective provided chemical dosages are reasonable. Section 2.9.1 presents the case history of Mine A which uses full scale in-pond chemical treatment. The results are positive and it is understood that the costs are reasonable.

3.1.11.1 Effectiveness of in-pond chemical treatment. In-pond chemical treatment is a less sophisticated version of mechanical treatment using chemicals (see Section 3.2.2). Experience with the physical-chemical treatment systems show that for in-pond chemical treatment to be successful, the system should consist of the following:

1. Chemical feed system;

2. Flow proportionate chemical dose system;
3. Chemical mixing and dispersion system;
4. Settling ponds modified to minimize sludge carry over;
5. Sludge removal and disposal facility; and
6. Power and fresh water.

The cost of installing and operating a proper system in remote areas can be high especially if fresh water and power are required. However, for a simple system, where the chemicals are fed neat at constant dosages and the ponds are used both as mixing and settling basins, the cost can be reasonable. Such a system can be effective but depending upon flow variations, sediment load, and sludge build up, it may be more susceptible to upsets. During winter periods, in-pond chemical treatment will be less effective because of slower chemical reaction rates and lower sediment-settling rates. These may be offset to some extent by lower wastewater flows. In general, the success of in-pond treatment is related to the ability to vary dosage to incoming flows, operator attention and regular desludging to remove the sludge.

3.1.12 Applicability to Alberta Foothills and Front Ranges

As stated previously, most augmentative technologies are, by nature, technologies used to control erosion and thus minimize the generation and transport of sediments. They are not end-of-the-pipe treatment or stand alone technologies. They can be very effective in minimizing generation of sediment when used at preplanning and active phases of a project. Once implemented, the technologies will help in long-term rehabilitation of disturbed areas.

The topography of mined areas in the Alberta Foothills and front ranges is often characterized by relatively steep slopes. The slopes influence the velocity of overland flows thus causing more erosion and transportation of sediment. Land treatment, mulches, diversions, vegetative filter strips etc. are technologies which can be applied successfully in the foothills and front ranges. However, for most successful implementation, augmentative technologies have to be built into the mine plans. These technologies also present opportunities for existing mines but integration during active phase has to be assessed carefully.

The use of methods to improve settling pond performances have direct relevance to some of the existing sediment ponds in the foothills area. It may be possible to improve settling performance of some of the ponds by minimizing dead zones, reduce short circuiting and improve outflow from the ponds by using in-pond chemical treatment. These are site specific concepts and should be evaluated in details before implementation.

3.2 ALTERNATIVE TECHNOLOGIES

3.2.1 Introduction

This section will examine some of the technologies that could be used as 'stand-alone' alternatives to sediment removal by ponding. Table 48 shows those separation technologies that were considered to have some potential for disturbed area sediment trapping applications. The table also shows the particle size and solids concentration ranges in which each technology is normally applied. From this original group of technologies, the following short list was prepared:

1. Chemical treatment plant;
2. Centrifugation;
3. Vacuum filtration;
4. Hydrocyclones;
5. Ultrafiltration;
6. Cross flow microfiltration; and
7. Electrokinetic methods.

In preparing this short list, an attempt was made to emphasize those technologies with the capability to separate slurries with particle size distributions in the finer end of the range normally encountered in drainage from disturbed areas. This was done because it is in this relatively fine range of particle size distributions that the

Table 48. Normal operating ranges for alternative sediment removal technologies.

Technology	Normal Range of Particle Sizes Removed (microns)	Normal Range of Solids Concentrations Treated (mg/L)	Reference
1. Chemical Treatment Plant	0.5 - 1000[a]	0 - 10 000	-
2. Centrifugation	4 - 1000[a]	10 000 - 500 000	Svarovsky 1979a; Ballew 1978
3. Vacuum Filtration	5 - 1000[a]	10 000 - 500 000	Roe 1983; Ballew 1978
4. Hydrocyclones	10 - 1000[a]	1000 - 20 000	Svarovsky 1979b
5. Microscreens	25 - 1000[a]	0 - 100	Culp, Wesner, Culp 1978; USEPA 1975

continued...

Table 48. Continued.

	Technology	Normal Range of Particle Sizes Removed (microns)	Normal Range of Solids Concentrations Treated (mg/L)	Reference
6.	Trommel Screen	100 - 1000[a]	10 000 - 500 000	Roe 1983
7.	Cross-Flow Microfiltration	0.1 - 10	0 - 100 000	Schneider and Klein 1982; Neptune Microfloc 1983
8.	Ultrafiltration	0.002 - 10	0 - 100	Ballew 1978
9.	Reverse Osmosis	0.0004 - 0.06	0 - 50	Ballew 1978; Schneider and Klein 1982
10.	Disposable Cartridge Filters	0.01 - 10	10 - 100	Ballew 1978

continued...

Table 48. Concluded.

Technology	Normal Range of Particle Sizes Removed (microns)	Normal Range of Solids Concentrations Treated (mg/L)	Reference
11. Granular Media Depth Filtration	0.1 - 100	0 - 200	Wakeman 1979
12. Thermal Dewatering	0.001 - 1000[a]	0 - 500 000	Roe 1983
13. Electrokinetic Methods	0.001 - 10	0 - 750 000	Dorr - Oliver 1984
14. Spray Irrigation	0.1 - 1000[a]	0 - 10 000	-

[a] Upper limit specified is approximate only. In some situations larger maximum particle sizes may be tolerated.

traditional settling pond technology is least effective, consequently, it is the range for which alternatives are most likely to be sought.

The technologies that were not included in the above list are shown in Table 49 along with a brief description of the criteria used to reject them. The fact that a particular technology is not included in the short list should not be taken as a definitive indication that it would be inappropriate for application in any particular disturbed area. It is conceivable that an unusual combination of site-specific factors would dictate the use of one of these systems.

It should be recognized that most of the technologies described in the following sections have rarely, if ever, been used for the application under consideration. Therefore, the fact that a particular system is included in the discussion should not be misconstrued as a tacit guarantee of technical feasibility. Most of the following technologies would have to be proven in pilot-scale field studies before full-scale installations could be considered.

It should also be noted that all of the alternative technologies discussed in subsequent sections have a common requirement for a heated building enclosing all equipment and supplies. Building size requirements may vary, but the relatively complicated nature of all these technologies demands that above-freezing temperatures be maintained if reliable operation and consistent effluent quality is to be assured under winter conditions.

3.2.2 Chemical Treatment Plant

3.2.2.1 Description. Treatment of disturbed area runoff in a chemical treatment plant would be similar conceptually to the lagoon chemical treatment technology described in Section 3.1.11. In both cases, a chemical is added to the wastewater to flocculate the incoming solids so that during a subsequent period of quiescent detention they may be easily separated by gravity from the feedwater. The major difference in the two systems lies in the efficiency with

Table 49. Rejected alternative technologies.

	Technology	Rationale for Rejection in Disturbed Area Sediment Removal Applications
1.	Microscreens	Not suitable for concentrated influents with fine particle size distributions.
2.	Trommel Screen	Not suitable for fine particle size distributions.
3.	Reverse Osmosis	Not suitable for high suspended solids concentrations.
4.	Disposable Cartridge Filters	Not suitable for high suspended solids concentrations.
5.	Granular Media Depth Filtration	Not suitable for high suspended solids concentrations.
6.	Thermal Dewatering	High energy costs.
7.	Spray Irrigation	High land requirements.

which the various physical processes involved are carried out. In a chemical treatment plant the mechanisms involved in floc formation are much more controlled and therefore it is easier to maximize the size and quality of floc that can be produced from a given chemical dosage. This improvement in floc quality, in conjunction with the optimized flow patterns that are possible in a chemical treatment plant, allows for the effective separation of wastewater solids at overflow rates (daily throughput per unit of available settling area) much higher than those typically seen for lagoon treatment systems.

A chemical treatment facility for the removal of sediment from disturbed area runoff would probably include the following major components:

1. Solids-contact clarifiers,
2. Chemical solution mixing and feed systems,
3. Bulk chemical storage facilities,
4. Potable water delivery system, and
5. Effluent pH adjustment system.

The clarifiers would represent the heart of any runoff chemical treatment plant. Clarifiers are rectangular or cylindrical vessels in which upflow velocities and detention times are such that the smallest flocculated particle is removed from solution before being discharged. There are three basic processes involved in successful clarifier solids - liquid separations:

1. Mixing,
2. Flocculation, and
3. Clarification.

In the mixing process, the chemical or combination of chemicals used for flocculation is rapidly dispersed throughout the wastewater, usually by violent agitation. During the flocculation phase, the water is agitated at lower velocities for a longer period of time to allow small particles to agglomerate into well-defined, settleable flocs. The flocs are then transferred to a zone of quiescent settling where they are retained long enough for the agglomerated solids to settle to a sludge blanket.

In solids-contact clarifiers, the mixing, flocculation, and clarification processes are combined in a single unit designed so that a large volume of previously formed floc is retained in the system. This high floc volume is maintained by continuously recirculating previously precipitated material from the clarification to the flocculation zone. These recirculating precipitates provide a relatively large surface area on which new precipitates can form, thereby substantially accelerating the flocculation process. Flow recirculation is provided by a low rpm turbine that also provides the velocity gradients necessary for rapid mixing and flocculation. Sludge is collected at the bottom of the clarifier and consolidated by the action of a slowly rotating rake. The sludge is partially removed from time to time to maintain the sludge inventory and recirculating slurry solids concentration at optimum levels. A typical solids-contact clarifier is shown in Figure 73.

The chemical feed system is used to inject the appropriate dosage of chemical(s) into the clarifier's rapid mixing zone. The scale of the feed system would vary as a function of the dosage requirements of the wastewater in question. Concentrated runoffs with very fine particle size distributions would require high chemical dosages and correspondingly large chemical feed systems. In addition, these difficult-to-treat runoffs would typically use more than one chemical and would therefore require a two- or possibly three-stage chemical feed facility.

The chemicals used in this type of treatment facility are almost always most effective when diluted with large proportions of potable water. Consequently, a treatment plant would need a source of potable water with a volumetric capacity defined by the chemical dosages required for effective floc formation. Mixing facilities would also be necessary for combining the appropriate volumes of chemical and water to be used in the chemical feed system.

If the characteristics of the wastewater are such that high chemical dosages are necessary, it is usually cost effective to use influent and effluent pH adjustment systems. Most chemicals have an

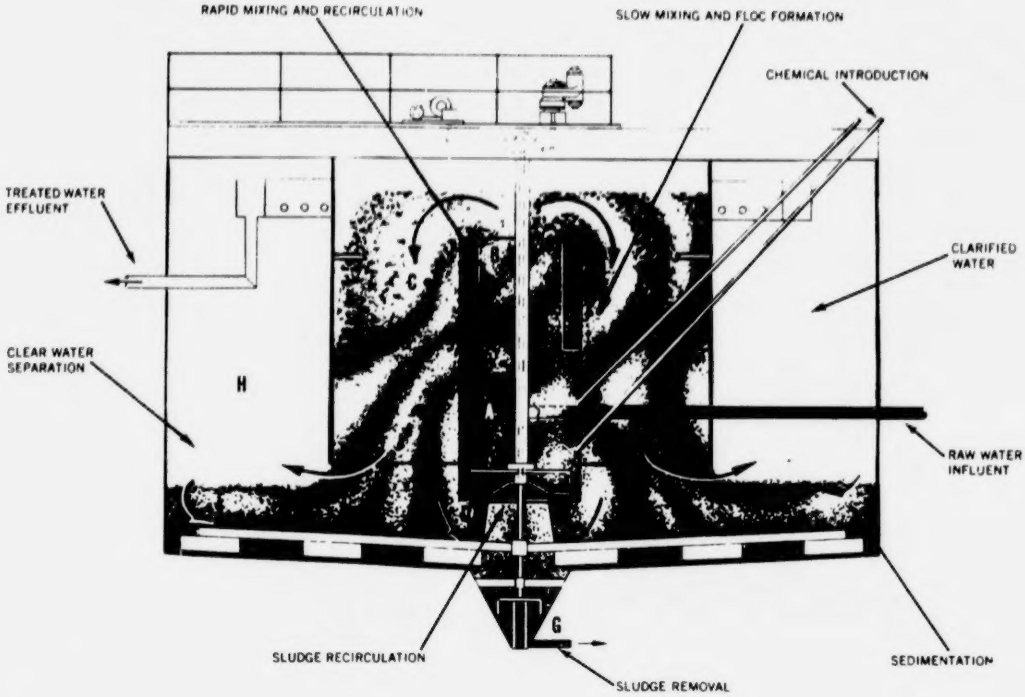

RAPID MIXING AND RECIRCULATION
SLOW MIXING AND FLOC FORMATION
CHEMICAL INTRODUCTION
TREATED WATER EFFLUENT
CLARIFIED WATER
CLEAR WATER SEPARATION
H
C
B
A
RAW WATER INFLUENT
G
SLUDGE RECIRCULATION
SLUDGE REMOVAL
SEDIMENTATION

Figure 73. Typical solids-contact slurry recirculation clarifier. (Graver Water 1982)

optimum pH range for flocculation and by adjusting the influent pH to be within this range, chemical consumption can be minimized. If the pH adjustment drops the effluent pH below the minimum established by the regulatory authority (usually about 6.5), an effluent pH adjustment system would be necessary.

Chemical treatment facilities of the type described above are capable of producing high quality effluent from a wide variety of wastewaters. If the nature of the chemical treatment regime is carefully matched to the feedwater characteristics, sediment particle diameters as small as 0.5 microns can be flocculated and separated. Figure 74 shows the process flow diagram for a facility that was designed to treat a runoff with high proportions of particle diameters between 0.5 and 5 microns at total suspended solids concentrations of up to 10 000 mg/L.

3.2.2.2 Traditional applications. Chemical treatment plants have been used for many years in a wide variety of water and wastewater treatment applications. Generally speaking, these kinds of facilities have been used wherever it is necessary to separate from solution a solid with a density higher than the water in which it is suspended. Typical applications include:

1. Removal of turbidity and hardness in the production of potable water;
2. First step in the production of high quality water for use as make-up in boiler feedwater systems;
3. Sewage treatment; and
4. Treatment of process wastewater in the mineral processing industries.

3.2.2.3 Advantages and disadvantages. The major advantage offered by a chemical treatment plant is its ability to produce high quality effluent over a wide range of feedwater sediment particle size distributions. High proportions of particles smaller than five

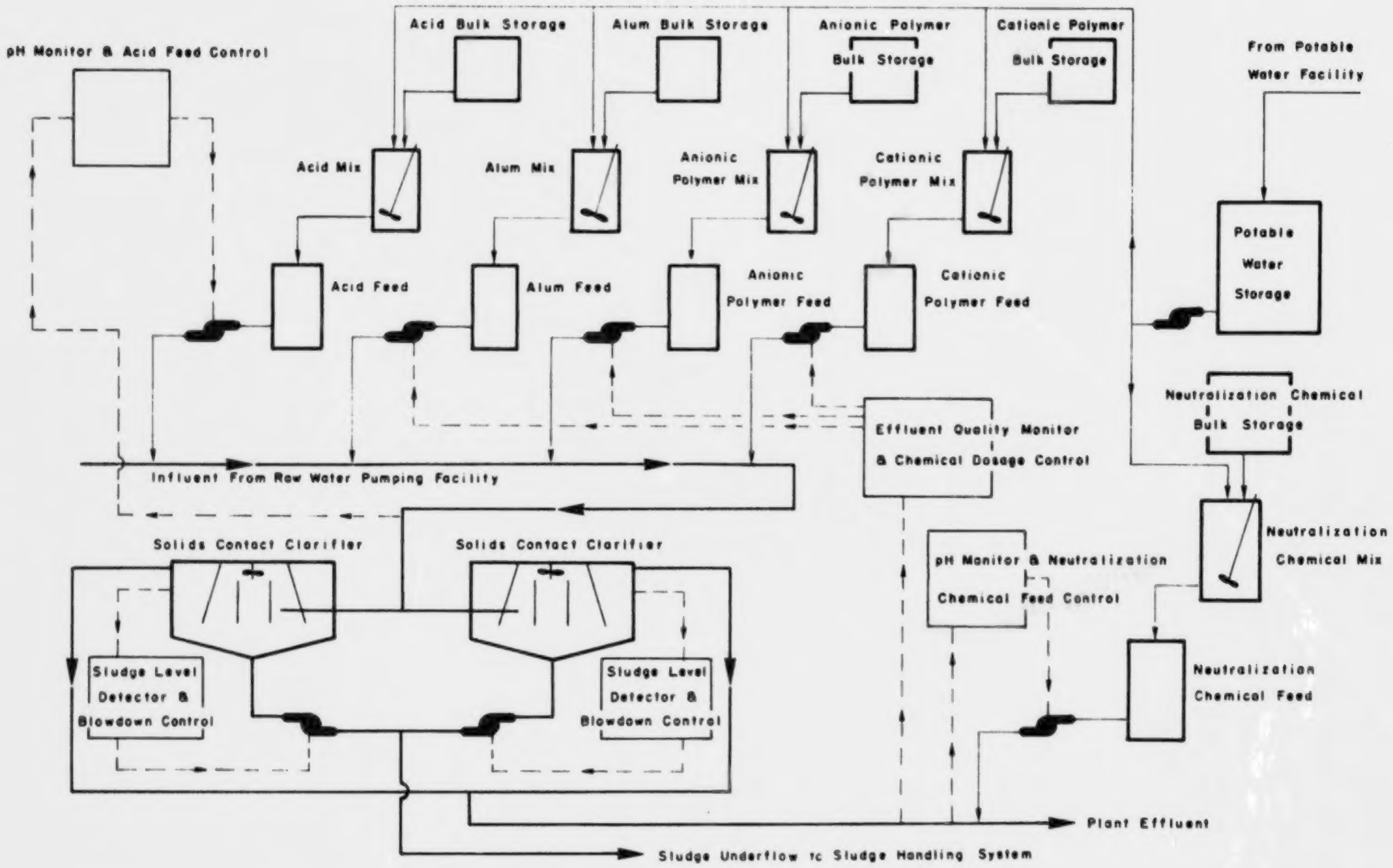

pH Monitor & Acid Feed Control
Acid Bulk Storage
Alum Bulk Storage
Anionic Polymer Bulk Storage
Cationic Polymer Bulk Storage
From Potable Water Facility
Acid Mix
Alum Mix
Anionic Polymer Mix
Cationic Polymer Mix
Potable Water Storage
Acid Feed
Alum Feed
Anionic Polymer Feed
Cationic Polymer Feed
Neutralization Chemical Bulk Storage
Effluent Quality Monitor & Chemical Dosage Control
Influent From Raw Water Pumping Facility
Neutralization Chemical Mix
Solids Contact Clarifier
Solids Contact Clarifier
pH Monitor & Neutralization Chemical Feed Control
Sludge Level Detector & Blowdown Control
Sludge Level Detector & Blowdown Control
Neutralization Chemical Feed
Plant Effluent
Sludge Underflow to Sludge Handling System

Figure 74. Typical chemical treatment design concept for runoff sediment removal applications.

microns in diameter need not reduce the quality of effluent if chemical dosages are properly matched to chemical demand.

Balancing this primary advantage is the following list of disadvantages associated with chemical treatment facilities:

1. Require a relatively high level of operational input and expertise for consistent effluent quality;
2. Chemical costs can be very high when treating wastewaters with high concentrations of sediment particles smaller than five microns;
3. Limited to relatively low influent suspended solids concentrations (approximately 5000 to 10 000 mg/L) if acceptable effluent quality is to be maintained;
4. Require a comparatively high capacity source of potable water; and
5. Space requirements are high relative to other alternative technologies.

3.2.2.4 Design and operational considerations. The following parameters are those that require the most consideration during the design of a chemical treatment facility:

1. Clarifier overflow rate,
2. Number and type of chemicals used,
3. Chemical dosages,
4. Sludge characteristics, and
5. Recirculation turbine speed.

A clarifier's overflow rate is a measure of its throughput per unit of available settling area and is usually reported in L/d/m^2. Overflow rates for solids-contact clarifiers typically range between 25 000 L/d/m^2 and 50 000 L/d/m^2. The overflow rate is directly related to the upflow velocity in the clarification zone and therefore has a profound effect on the minimum particle size that eventually drops to the sludge blanket. The American Water Works Association (1969) suggests that the overflow rate be selected so that the upflow velocity is at least 50% below the minimum particle settling velocity at a level 1.5 m below the point of effluent takeoff. The problem with this approach is that it is usually very difficult to predict minimum particle sizes when chemical flocculants are used. There are no practically applicable analytical procedures that can be used to predict floc size as a function of chemical dosage and therefore, for a given wastewater, this relationship must be established on the basis of jar test or pilot studies. Of the two methods, pilot studies are much more desirable since machine operational variables are evaluated in addition to the relationship between floc size and chemical dosage.

It should be noted that the above approach establishes a maximum allowable overflow rate without any consideration of the clarifier's solids loading. The assumption has been made that influent sediment concentrations are sufficiently dilute to avoid the problem of solids buildup in the clarifier. When dealing with very concentrated solutions, a method that calculates the maximum allowable solids loading must be used. Detailed descriptions of the required methodology can be found in USEPA (1975) and Clark, Viessman, and Hammer (1977).

The chemicals that are most commonly used in disturbed area sediment removal applications are anionic and cationic polymers. Polymers are long-chain, charged molecules that promote flocculation by charge satisfaction and interparticle bridging (see Section 2.6.1.1.2). Polymers are prepared for use by diluting them with water to a concentration recommended by the manufacturer. Anionic polymers are typically mixed and used at concentrations between 0.4% and 1.0% by weight while cationic polymers are normally mixed at a more concentrated 5% to 10% by weight.

The required polymer dosage varies widely between applications and is primarily dependent on influent suspended solids concentrations and the proportion of the total solids content made up of very fine particles between 0.5 and 5 microns in diameter. Other factors, such as wastewater alkalinity and positive ion concentration, may have an influence on chemical demand, but this impact is not as profound as that of the physical characteristics of the sediment. Although polymers are generally effective over a relatively wide range of pH, consumption can often be reduced by adjusting the feedwater pH to an optimum range over which polymer effectiveness is maximized. The optimum pH for a particular polymer can vary among wastewaters, but for anionic polymers it normally ranges between 5.0 and 7.0, while for cationic polymers, best results are typically observed between pH levels of 3.0 and 7.0.

The methods used to handle the sludge produced in a treatment facility can have a substantial impact on both the performance of the clarifiers and the overall cost of the treatment operation. If the rate of sludge transport (blowdown rate) is too low, solids will accumulate in the clarifier and eventually begin to carry over into the plant effluent. If the sludge blowdown rate is too high, the solids inventory will be reduced to the point where insufficient material is available for recirculation. In addition, the sludge produced will be of relatively low density, increasing downstream sludge handling and disposal costs. A plant with a properly monitored sludge blowdown system will typically produce sludge densities ranging from 4% to 8% solids by weight.

The turbine in a solids-contact clarifier is used to recirculate previously precipitated solids and to provide the velocity gradients necessary for rapid mixing and flocculation. The turbine speed is adjustable so that the rate of recirculation can be varied to suit the particulars of the wastewater being treated. Recirculation flow volumes usually vary between three and five times the clarifier's design throughput volume. The maximum allowable turbine speed is usually defined by the maximum liquid velocity gradient that does not cause flocculated material to break up. These floc shear problems can usually be avoided by limiting peripheral turbine speeds to 1.5 m/s (American Water Works Association 1969).

3.2.2.5 Suitability for sediment removal applications in the Alberta foothills and front ranges. A properly monitored chemical treatment facility has the capability to produce effluent of consistently high quality under varying influent and weather conditions. It is very likely that the effluent produced could be discharged directly to the nearest watercourse without any further treatment. In addition, sludge densities could be expected to be higher than those achievable with a lagoon chemical treatment operation (Section 3.1.11).

One of the major difficulties with chemical treatment plants for disturbed area sediment removal applications is that their chemical costs are likely to be high when treating runoff with heavy concentrations of particles smaller than five microns in diameter. Unfortunately, it is this range of influent particle sizes for which alternatives to the traditional sediment pond technology are most likely to be sought. Therefore, before a chemical treatment plant is proposed as an alternative for dealing with a difficult-to-treat runoff, it would be advisable to generate reliable chemical cost estimates using pilot-scale field data.

3.2.3 Centrifugation

3.2.3.1 Description. Centrifugation is a process that uses density differences to separate solids from liquids. The incoming wastewater is subjected to centrifugal forces that make the solids move radially through the liquid: outward if heavier, inward if lighter. Centrifugation is actually analogous to the extension of gravity sedimentation to relatively fine particle sizes (Svarovsky 1979b).

There are a wide variety of centrifuges currently available on the market; however, only a few of these are likely to have any potential for disturbed area sediment removal applications. After reviewing the literature, it was determined that the solid-bowl, sedimenting-type centrifuge was representative of those centrifuges that appear to have some potential for this application. A sedimenting–type centrifuge is one in which the separation process relies only on the density difference between the solid and liquid phases and does not make use of filtration through any semi-permeable membrane. Solid-bowl, scroll, or decanter sedimenting centrifuges consist of a horizontally rotating chamber that has one end tapered into a cone. The wastewater is admitted through axial feed tubes and removed radially out of the bowl. A screen or scroll mechanism rotates in the same direction as the bowl, but at a slightly different speed so that it can push the solids along the length of the chamber. The solids collected by this scroll are directed towards the tapered end of the bowl, while the solution overflows a weir at the other end. The speed of the bowl rotation usually varies from about 1600 to 6000 rpm (Svarovsky 1979b). The tapered section of the bowl serves as a drying zone or beach area for the solids prior to their discharge from the machine (Pollution Equipment News 1980; Svarovsky 1979d, cited by Roe 1983). Figure 75 shows a simplified cross-section of a typical solid-bowl centrifuge.

Solid-bowl or decanter centrifuges are capable of removing approximately 95% of particles larger than four microns in diameter. Figure 76 shows the grade efficiency curve for this type of centrifuge along with a number of other solid-liquid separation devices. In Figure 77, a parameter known as the X_{50} cut size is shown in relation to the incoming wastewater flow rate and solids concentration. The X_{50} cut size is defined as that particle diameter for which the cumulative solids recovery is 50% (Svarovsky 1979a).

3.2.3.2 Traditional applications. The solid-bowl centrifuge has a wide variety of industrial applications both as a clarifier and as a sludge dewatering device. As a clarifier it has been used to recover crystals and polymers, and as a dewatering device it has been used to densify a number of industrial and municipal sludges (Svarovsky 1979b). Specially designed models of solid-bowl centrifuges have been used downstream of hydrocyclones (see Section 3.2.5) for the recovering and dewatering of deslimed coal (Roe 1983). Another major

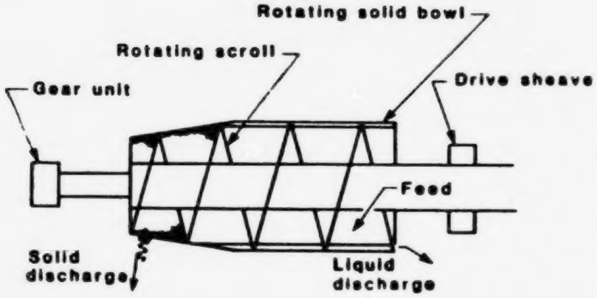

Rotating solid bowl
Rotating scroll
Gear unit
Drive sheave
Feed
Solid discharge
Liquid discharge

Figure 75. Cross-section of a solid-bowl centrifuge. (Roe 1983)

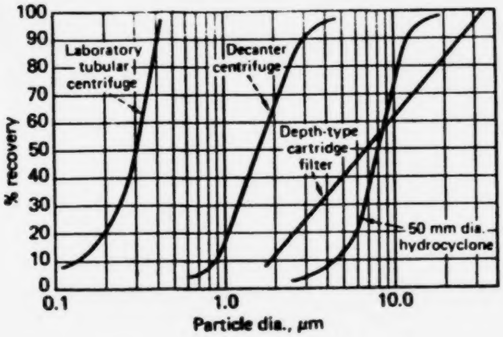


Figure 76. Grade efficiency curves for some common solid-liquid separation devices.(Svarovsky 1979a)

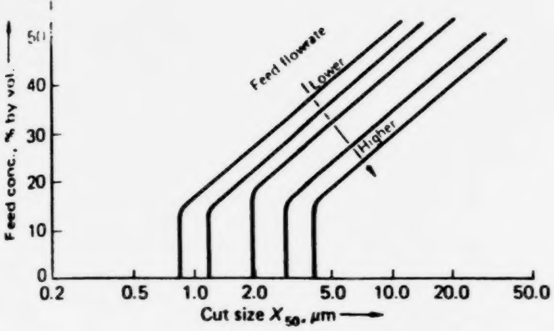


Figure 77. Effect of feed concentration and flow rate on cut size X_{50} for a scroll-type centrifuge. (Svarovsky 1979b)

area of application is in the classification of solids such as kaolin coating clay and titanium oxide (Svarovsky 1979b).

3.2.3.3 Advantages and disadvantages. The main advantage of a centrifuge is its operational flexibility. Within its design limits, a centrifuge can be fed a wastewater at various rates and still produce a reasonably consistent effluent and sludge quality. If the feed rate exceeds the design limits of the machine, excess solids begin to appear in the effluent; however, the quality of the discharged cake does not deteriorate (Envirotech Corp. 1979, cited by Roe 1983).

A major disadvantage of centrifuges is the high operating and maintenance cost associated with them. Centrifuges have high energy requirements and are subject to abrasive wear on the scroll and other interior parts. Construction materials must be selected by considering the abrasive or corrosive characteristics of the incoming wastewater if maintenance difficulties are to be minimized (Roe 1983).

Another drawback of centrifugation is that the wastewater may need to be prethickened for efficient machine operation. Although centrifuges can separate a wide range of wastewaters, a low feed solids content (less than about 20 000 mg/L) requires the centrifuge to process large volumes of inflow. Since the capacity of a centrifuge is limited by its maximum allowable volumetric throughput, a relatively low feed solids concentration would require a proportionately large number of machines. It is usually more cost effective to prethicken the influent by sedimentation, hydrocycloning, or some other means prior to centrifugation.

3.2.3.4 Design and operational considerations. The performance and efficiency of a centrifuge depends on a number of factors, the most important of which are (Envirotech Corp. 1979, cited by Roe 1983):

1. Rotation speed,
2. Diameter,
3. Length,
4. Beach angle and length,

5. Wastewater feed point,
6. Flocculent feed point (if used),
7. Scroll rotational speed differential and pitch, and
8. Pool depth.

The efficiency of solids recovery in a centrifuge is closely related to the rotational speed of the bowl. Generally speaking, an increase in the rotational speed results in a higher solids recovery. However, higher speeds also increase both the maintenance and operating costs of the centrifuge.

Bowl diameters range from 150 mm to 1300 mm, while bowl lengths vary from two to four times the bowl diameter. Bowl speed is usually correlated to bowl diameter since it is the combined effect of speed and diameter that establishes the magnitude of the centrifugal force acting on the wastewater. The centrifugal force typically ranges from 1000 to 4000 times the force of gravity, with the higher forces associated with the smallest bowl diameters (Envirotech Corp. 1979, cited by Roe 1983).

The bowl length has a significant effect on the centrate (effluent) clarity. The residence time of the wastewater increases with bowl length, allowing for greater retention of the finest sediment particles (Baker 1979, cited by Roe 1983).

The density of the final cake and the torque required to move the solids out of the centrifuge are functions of the beach angle and length. The longer the cake is allowed to dewater on the beach, the higher the torque requirements for discharge.

The point at which the wastewater is injected into the bowl influences both the solids recovery and cake density. Solids recovery is usually improved if the feed point is in the beach area, since the sediment particles then have a longer distance to migrate to the effluent discharge end of the machine (Envirotech Corp. 1979, cited by Roe 1983).

Flocculants (e.g., metal salts, polymers) are often injected into the wastewater to enhance the centrifuge's separation efficiency for fine particles. The required flocculant dosage would be defined by the characteristics of the wastewater and the desired increase in centrifuge throughput and/or centrate quality. The point of flocculant addition varies depending on the nature of the chemical used and the quality of the influent. Anionic polymers are normally injected upstream of the centrifuge, while the more rapidly acting cationic polymers are usually added inside the machine (Svarovsky 1979d, cited by Roe 1983).

The magnitude of the scroll differential rotational speed affects the particle residence time and thus influences both solids recovery and cake density. If it is desirable to maximize the recovery of fine particles, the scroll differential rotational speed should be minimized.

The difference between the bowl radius and the radius to the inner surface of the rotating liquid is referred to as the pool depth. In most solid-bowl centrifuges the pool depth is adjustable so that the wastewater retention time can be varied. Greater pool depths (i.e., longer retention times) improve the separation efficiency of the machine at the expense of throughput (Envirotech Corp. 1979, cited by Roe 1983).

3.2.3.5 Suitability for sediment removal applications in the Alberta foothills and front ranges. The mechanical complexity and ancillary service requirements of centrifuge installations limit their applicability to areas or operations with unusual site-specific conditions. A centrifuge installation would require high levels of operational input and expertise and would most likely have to be preceded by some sort of prethickener for cost-effective operation. Wastewaters with high concentrations of particles smaller than five microns in diameter would probably require the use of polymers for acceptable effluent quality.

The above comments notwithstanding, it is possible that centrifugation could be a desirable alternative for watersheds discharging a concentrated wastewater with a relatively coarse particle size distribution if topographic constraints are such that settling pond construction either prohibitively expensive or impossible. The relatively high sludge density produced by a centrifuge installation would also be an advantage in areas where the space available for sludging handling, dewatering, and disposal facilities is limited.

3.2.4 Vacuum Filtration

3.2.4.1 Description. Filtration is a process in which a wastewater is passed through a permeable medi has the capacity to retain solids. In order to maintain the flow of liquid through the medi pressure drop must be applied. In the case of vacuum filtration the pressure applied is lower than atmospheric and is induced downstream of the filter media. The vacuum filtration devices that will be described in this section are classified as surface, as opposed to depth filters. Surface filters are used for cake filtration in which the solids are deposited in the form of a cake on the surface of a relatively thin filter medium. Depth filters (e.g. mixed granular media filters) make use of deep-bed filtration wherein the particles are deposited within the filter medium (Svarovsky 1979a).

In the surface filtration process, the sediment particles that are the same size or larger than the media openings are the first to be retained. As more of these coarse particles are retained, a filter cake is formed which in turn, functions as a filter for the finer particles in the incoming wastewater. The filter cake is then mechanically removed and the filter media cleaned prior to being returned to a position to receive more solids. The performance of a vacuum filter is usually expressed in terms of its yield in kilograms of dry solids per square metre of filter area per hour (Clark, Viessman, and Hammer 1977).

The filter medium is the single most important component of the filter that affects its performance. Filter media can be composed of duck, chain, and twill weaves, as well as felt made of cotton, nylon, polyester, polypropylene, or other natural or synthetic materials. Filter cloths and screens made of steel, stainless steel, and other alloys are also available. If a high solids retention capacity is required, paper filters are often used. Filter paper requires the support of filter cloth, screens, or perforated metal sheets, and is usually manufactured so that it can be easily removed from the support filter medium (Sperry undated, cited by Roe 1983:22).

From the various types of vacuum filtration units currently available, rotary drum vacuum filters were selected as being representative of the kind of device that might be suitable for disturbed area sediment removal. Drum vacuum filters are probably the most common type of continuous vacuum filter presently in use. They consist of a horizontal cylinder or drum, whose circumference supports the filter medium. This drum rotates in a slurry tank, and a vacuum inside the drum draws the wastewater through the unit, leaving the solids captured on the filter surface. A scraper, or some other mechanical device, removes the cake from the drum and the medium re-enters the tank (Pollution Equipment News 1980, cited by Roe 1983:26). Figure 78 shows a cross-section of a typical rotary drum vacuum filter.

A single rotary vacuum drum can provide volumetric throughputs of up to 1000 L/min at an average yield of between 9 and 65 kg of dry solids per square metre per hour (Clark, Viessman, and Hammer 1977). However, without a filter aid, these devices are incapable of retaining particles smaller than about 50 microns in diameter. Consequently, for most disturbed area sediment removal applications, some sort of precoat would be required to prevent binding or clogging of the filter medium. A precoat consists of the application of materials such as diatomaceous earth, paper pulp, or perlite to the surface of the filter medium. Properly selected combinations of filter medium and precoat are capable of providing for the retention of particles as small as five microns in diameter (Sperry undated; Star Systems 1979, cited by Roe 1983).

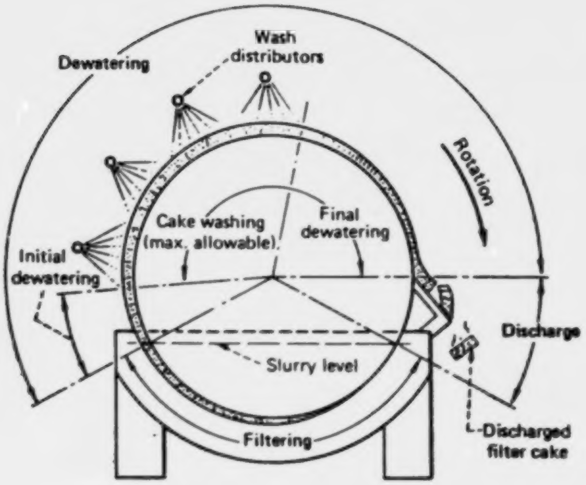

Wash distributors
Dewatering
Rotation
Cake washing (max. allowable)
Final dewatering
Initial dewatering
Discharge
Slurry level
Discharged filter cake
Filtering

Figure 78. Rotary drum vacuum filter. (Svarovsky 1979a)

3.2.4.2 Traditional applications. Vacuum filtration equipment has been used for many years in the mineral and chemical processing industries, primarily in sludge dewatering applications. The popularity of this equipment stems from its relatively simple operation and minimal auxiliary equipment requirements. Vacuum filters are not normally used as primary clarification devices because they require relatively concentrated influents and therefore, their use for disturbed area sediment removal would represent a fairly novel application.

3.2.4.3 Advantages and disadvantages. The use of rotary drum vacuum filters offers the following advantages (Svarovsky 1979d, McIlvaine Co. 1980, cited by Roe 1983):

1. Continuous rather than batch operation,
2. Design and operational flexibility which allows for handling a wide range of different influents,
3. Low filtrate suspended solids concentrations;
4. Clean operation;
5. Low maintenance requirement and cost; and
6. Does not require skilled personnel.

The disadvantages of rotary drum vacuum filtration are (Culp, Wesner, and Culp 1978; Svarovsky 1979a):

1. Dilute suspensions may require prethickening;
2. Particle sizes less than 50 microns necessitate precoat;
3. Filter medium tends to blind in some applications;
4. Very high energy requirement; and
5. Auxiliary equipment is very loud.

3.2.4.4 Design and operational considerations. The performance of vacuum filtration equipment is influenced by the following factors (Leonard and Mitchell 1968, cited by Roe 1983; Clark, Viessman, and Hammer 1977):

1. Feed solids concentration;

2. Cycle time;
3. Influent particle size distribution;
4. Filter medium;
5. Pressure drop across solids cake;
6. Cake thickness;
7. Permeability of cake;
8. Cake drying time;
9. Homogeneity of cake formation;
10. Specific gravity of dry solids;
11. Inherent moisture of dry solids;
12. Surface properties of solids;
13. Viscosity of filtrate; and
14. Surface tension of filtrate.

Of the above factors, probably the most important are feed solids concentration and cycle time. The rate of solids production in a vacuum filter is a measure of its efficiency and is closely related to a feed solids concentration. Most efficient operation is usually obtained when the feed solids concentration is maintained within an optimum range defined by the characteristics of the incoming wastewater. Figure 79 shows a solids production efficiency curve for a typical wastewater. For the example shown in the figure, some sort of prethickening device would probably be used to maintain the feed solids concentration between 45% and 55% solids by weight. This solids concentration range provides for the most effective use of the minimum possible filtration area. In the design of a sediment removal facility, a curve similar to Figure 79 would be used to define the most cost effective balance between prethickening and vacuum filtration capacity.

The other important consideration for filtration operations is cycle time which, for a continuous rotary vacuum filter, is defined as the time required for the filter to make one complete revolution. During each cycle, there are three phases of filter operation (Roe 1983:36).

1. Cake formation;

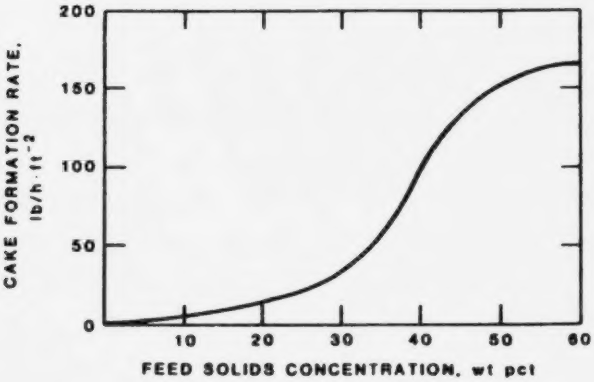


Figure 79. Representative curve for cake formation rate versus feed solids concentration.(Roe 1983)

2. Cake dewatering, and
3. Cake discharge.

At the end of each cycle, the filter discharges a certain amount of cake per given filter area. For an existing filtration installation, solids production data can be used to generate a log-log plot of dry cake formation rate versus cycle time for a given feed solids concentration. An example of such a plot is shown in Figure 80. Vacuum filters are normally equipped with variable speed drives, so that curves such as the one shown in Figure 80 can be used to establish the cycle time required for a desired rate of solids production.

Filter cycle time also affects the cake moisture content and dischargeability. Filtration equipment is normally sized on the basis of a minimum cycle time of 3 to 4 minutes per revolution. Cycle times less than 3 minutes per revolution result in increased cake moisture and produce thin cakes that are difficult to remove from the filter medium. Ongoing difficulties with cake removal can result in substantial increases in filter maintenance costs (Leonard and Mitchell 1968, cited by Roe 1983:36).

As noted previously, when sizing a full-scale vacuum filtration installation, one of the most important parameters that must be defined is the relationship between filter yield and feed solids concentration (Figure 79). This relationship is normally established on the basis of experience, filter-leaf laboratory tests, pilot-plant tests, or a combination of these (Clark, Viessman, and Hammer 1977). Of these methods, the most reliable results are probably those derived from pilot testing. Equipment manufacturers often lease portable pilot-plant filters that can be set up and run at the site of a proposed sediment removal installation.

3.2.4.5 Suitability for sediment removal applications in the Alberta foothills and front ranges. Vacuum filtration and centrifugation installations are similar in that both make use of relatively complicated equipment, would probably require upstream prethickening,

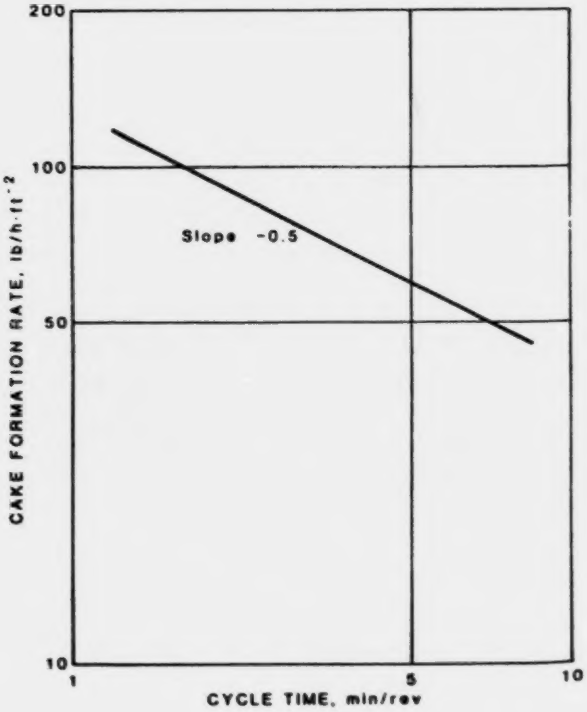


Figure 80. Representative curve for cake formation rate versus cycle time. (Roe 1983)

and produce a high sludge density. Consequently, the comments regarding centrifugation noted in Section 3.2.3.5 would also apply to vacuum filtration installations. One advantage that vacuum filtration would have over centrifugation is in the area of filtrate quality. Vacuum filters are more likely to produce an effluent quality which allows for direct discharge without any further polishing.

3.2.5 Hydrocyclones

3.2.5.1 Description. Hydrocyclones are devices which separate the solid and liquid phases using the principle of centrifugal sedimentation. Cyclones have no moving parts and the necessary vortex action is provided by the fluid itself.

A cyclone consists of a short cylindrical section, attached to an inverted, truncated, conical section. The apex or bottom of the conical section is called the underflow orifice. A central overflow orifice or vortex finder is fitted to the base of the cone, and a feed orifice is attached tangentially to the cylindrical body section. Figure 81 shows the cross section of a typical hydrocyclone. The slurry enters at high pressure through the tangential feed orifice into the cylindrical section, where a rotating force field is established. The solids in the slurry are settled to the side wall by this force, slide down the inclined wall to the apex of the cone, and exit through the underflow orifice. The liquid portion of the feed travels to the center of the cone with some of the finest solids and exits through the overflow orifice. A vortex of air extends throughout the length of the cyclone (Roe 1983:37).

3.2.5.2 Traditional applications. Hydrocyclones have been applied to the cleaning of cooling oils, separation of mineral products, regeneration of drilling mud, cleaning of air scrubber water, and for the separation of two immiscible liquids (Svarovsky 1979b). Hydrocyclones, by themselves, are not capable of producing a dry solids

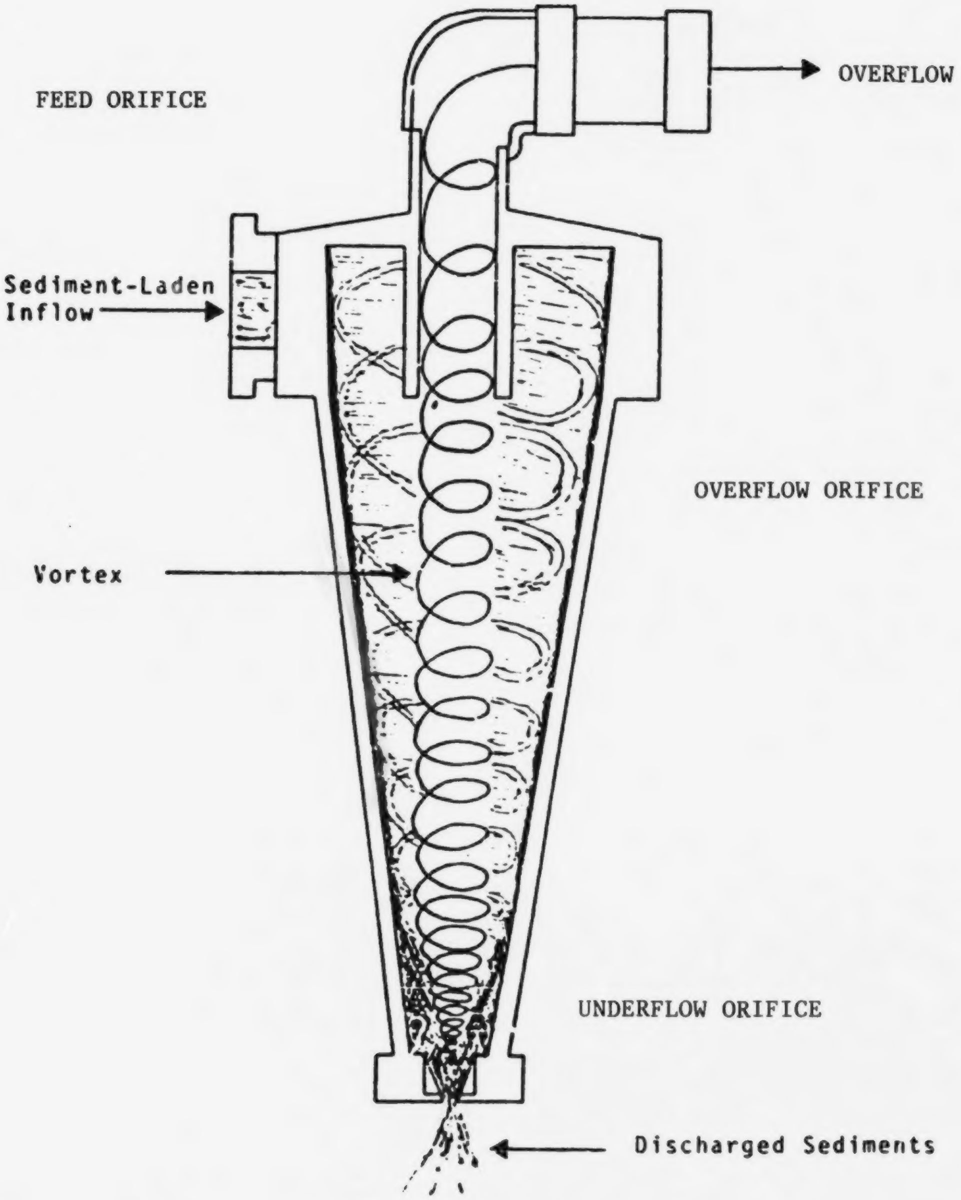

OVERFLOW
FEED ORIFICE
Sediment-Laden Inflow
OVERFLOW ORIFICE
Vortex
UNDERFLOW ORIFICE
Discharged Sediments

Figure 81. Cross-section of a hydrocyclone. (Nawrocki and Pietczak 1976)

product and therefore they are often used upstream of another dewatering device such as a vacuum filter or centrifuge.

3.2.5.3 Advantages and Disadvantages. Hydrocyclone solid-liquid separation installations have the following advantages (Roe 1983):

1. High capacity;
2. Simple operation;
3. Compact;
4. Low capital cost; and
5. Low operating and maintenance cost.

The major drawback of hydrocyclones is their limited ability to remove particles smaller than about 10 microns in diameter from the incoming flow. Proper cyclone design can maximize fines removal, however, subsequent clarification of the overflow may be necessary when treating flows with a high proportion of very fine particles.

3.2.5.4 Design and operational considerations. Although a hydrocyclone is a relatively simple device, its principles of operation are complex and there are a number of variables that must be considered in its design. The most significant of these variables are (Roe 1983):

1. Cyclone diameter;
2. Cyclone cone angle;
3. Feed, overflow, and underflow orifice sizes;
4. Length of cylindrical section;
5. Feed pressure;
6. Feed concentration; and
7. Particle size.

Cylinder diameter is probably the most important factor influencing hydrocyclone design because the smaller the particle to be removed, the smaller the cyclone diameter that must be used. For most applications, the cyclone manufacturer determines the size and cone angle as a function of the characteristics of the wastewater to be treated (Leonard and Mitchell 1968, cited by Roe 1983).

The various orifice sizes also have a profound impact on hydrocyclone performance. The diameter of the underflow orifice determines the concentration and flow of the sludge discharged from the cyclone. Enlarging the underflow orifice increases the flow rate and the percentage of fines in the underflow and also decreases its solids concentration. A larger overflow orifice increases the throughput, the concentration of solids, and the maximum particle size contained in the cone overflow. Increasing the overflow orifice diameter also results in a higher underflow solids concentration and percentage of coarse particles. A larger feed orifice diameter increases the flow to the cyclone and reduces the retention time of the wastewater, causing the cyclone to reject coarser materials to the overflow. Consequently, when the influent flow is increased, the diameter of the underflow orifice should also be increased to handle the greater weight of sediment fed to the cyclone.

The feed pressure affects the volume of incoming flow as well as the efficiency of the cyclone. A higher feed pressure increases the underflow solids concentration and its proportion of fine particles. An elevated influent pressure also increases the overflow discharge and reduces its proportion of fine particles (Leonard and Mitchell 1968, cited by Roe 1983).

When planning a hydrocyclone installation, the designer must select the size and number of cyclones required as a function of the flow rate and the desired separation efficiency. After the volume and solids characteristics of the wastewater have been identified, the maximum available pressure differential between the inlet and outlet orificies is defined. With this information, the manufacturer's operational data is reviewed to determine the optimum characteristics and configuration of the hydrocyclone installation. An example of the kind of manufacturer's information that would be used is shown in Figure 82. A more complete description of hydrocyclone design methodology can be found in Svarovsky (1979b).

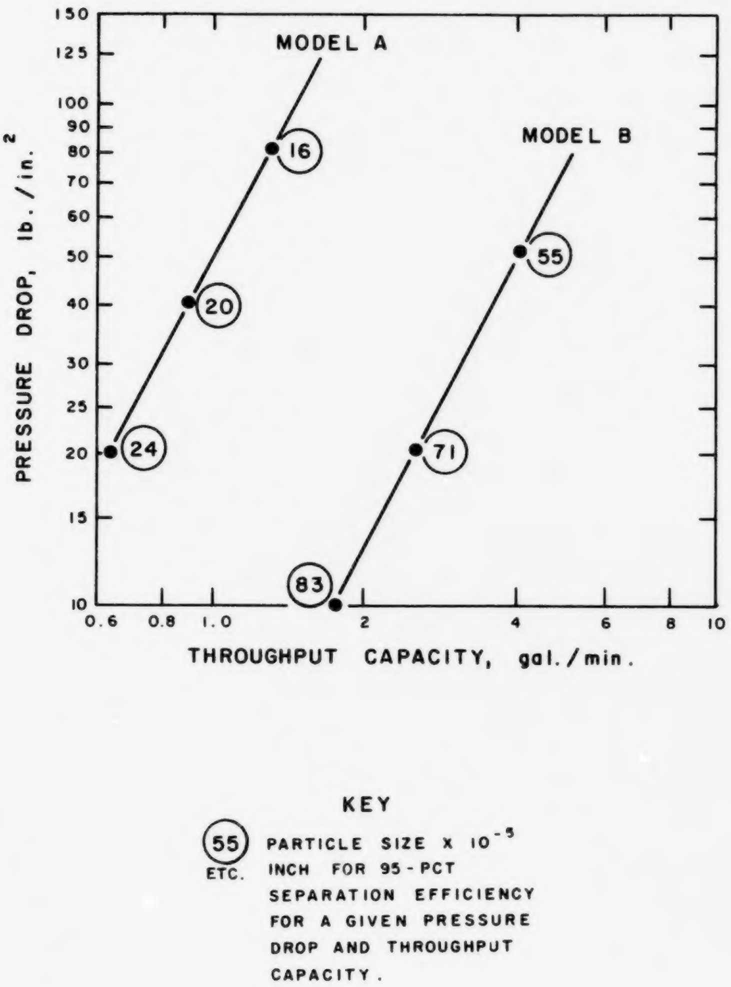


Figure 82. Graph of pressure drop versus throughput capacity for two hydrocyclones. (Roe 1983)

3.2.5.5 Suitability for sediment removal applications in the Alberta Foothills and front ranges. The relative simplicity of hydrocyclones would make them attractive alternatives in some situations. The machines themselves have no moving parts and although inlet pumping facilities would be required, overall operating and maintenance requirements would be relatively modest. This would be a distinct advantage for applications in remote areas where maintenance is either difficult or very expensive to provide.

As noted previously, the major difficulty associated with hydrocyclones is their limited ability to remove fine particles between 1 and 10 microns in diameter. In addition, there was little quantitative information in the literature on the effluent and sludge quality that could be expected with a hydrocyclone. It is possible therefore, that a hydrocyclone installation for disturbed area sediment removal would still require small lagoons for effluent polishing and/or sludge storage and dewatering.

3.2.6 Ultrafiltration

3.2.6.1 Description. Ultrafiltration is a membrane filtration technology that separates colloidal materials and high molecular weight solutes from their solvent by passing the wastewater through a relatively dense semi-permeable membrane under pressure. Ultrafiltration feedwater is pressurized by pumping and then channeled along the membrane surface. As product water (permeate) passes through the membrane, the feedwater becomes progressively more concentrated, requiring the maintenance of a continuous flow along the feedwater/concentrate side of the membrane to prevent an excessive buildup of solids (Tang, Chu, and Boroughs 1980:7). The ultrafiltration process is shown conceptually in Figure 83.

Because ultrafiltration is used to separate suspended and colloidal materials, both the membrane material and its physical configuration must be selected to facilitate cleaning. The wide array of

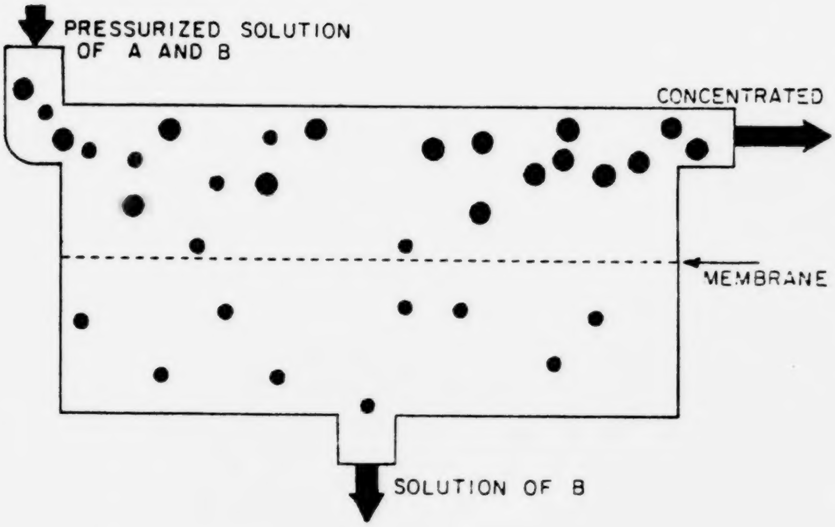

PRESSURIZED SOLUTION
OF A AND B
CONCENTRATED A
MEMBRANE
SOLUTION OF B

Figure 83. Schematic diagram of the ultrafiltration process. (Tang, Chu, and Boroughs 1980)

configurations currently available can be divided into four general groups (Horton 1980):

1. Tubular,
2. Flat plate,
3. Hollow fibre and
4. Spiral wound.

The tubular configuration usually consists of stainless steel membrane support tubes enclosing porous glass-fibre tubes. The frail ultrafiltration membranes are coated onto the inside of these fibre tubes. Groups of tubes are then connected in parallel as modules that physically resemble long, thin, single-pass shell and tube heat exchangers. The tubes are usually between 13 mm and 26 mm in diameter, easy to clean physically or chemically, and simple to remove from the modules (Svarovsky 1979c:75). Figure 84 shows a typical ultrafiltration module. Tubular configurations are normally used where difficulties are expected with membrane fouling. High fluid velocities and relatively large tube diameters combine to create a level of turbulence that scours the membrane walls (Tang, Chu, and Boroughs 1980:11).

The flat plate or 'plate and frame' configuration consists of a frame supporting a number of parallel sheets of membrane material. This frame also provides manifolds for distributing the wastewater and collecting the permeate. The permeate collection compartments alternate with feed-concentrate compartments (Tang, Chu, and Boroughs 1980). The plate and frame configuration was among the first to be employed on a commercial scale,but has since been largely superseded by other configurations. The plate and frame configuration is shown conceptually in Figure 85.

Hollow fibre ultrafiltration systems employ very small diameter tubes cemented at both ends into a wide plastic tube that serves as a receiver for the filtrate. With these systems a large membrane area can be packed into a small volume; however, the small tube diameter increases the susceptibility to fouling and the difficulties associated with cleaning. To mitigate these problems,

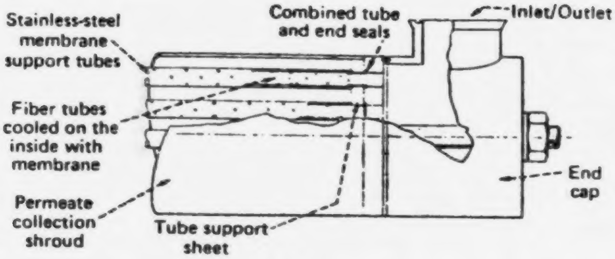

Stainless-steel membrane support tubes
Combined tube and end seals
Inlet/Outlet
Fiber tubes cooled on the inside with membrane
End cap
Permeate collection shroud
Tube support sheet

Figure 84. Tubular ultrafiltration module.(Svarovsky 1979c)

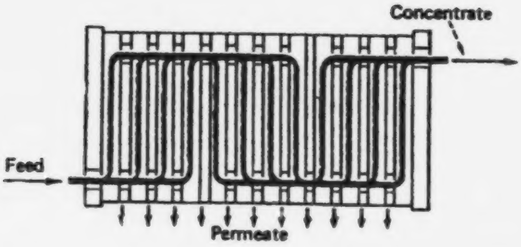

Concentrate
Feed
Permeate

Figure 85. Plate and frame ultrafiltration module. (Svarovsky 1979c)

some manufacturers are developing hollow fibres large enough to permit turbulent flow and rigid enough to permit backwashing (Svarovsky 1979c; Tang, Chu, and Boroughs 1980).

In the spiral-wound system, a large flat sheet of membrane is used to cover each side of a flat sheet of porous, water conducting backing material. The membrane is sealed with an adhesive on the two long edges and one end to form an 'envelope'. The open end of the assembly is sealed to a perforated permeate collection tube that runs longitudinally from one end of the ultrafiltration module housing to the other. A number of these envelopes are attached to the same permeate collection tube and then wound around the tube, separated from each other by a spacer. These spacers, usually consisting of an open mesh material, provide a path for the feed/concentrate cross-flow and are designed to promote the turbulence necessary to reduce fouling. Spiral-wound configurations have a higher membrane packing density (i.e., available filtration surface area) than tubular configurations,but are more prone to fouling (Tang, Chu, and Boroughs 1980; Ballew 1978).

A wide variety of membrane materials are presently available for commercial use. A membrane material is selected for a particular application on the basis of wastewater characteristics, temperature, and the nature of any chemical cleaning agents that are likely to be used. The ten most common membrane materials currently available are (Horton 1980):

1. Cellulose acetate (CA),
2. Chlorinated polyvinyl chloride (CPVC),
3. Polyamide (PA),
4. Polyacrylonitrile (PAN),
5. Polyimide (PI),
6. Polysulfone (PS),
7. Polyvinyl alchohol (cross-linked) (PVA),
8. Polyvinylidene fluoride (PVF),
9. Vinyl acrylonitrile copolymer (VAN), and
10. Zirconium (IV) oxide (Z_rO).

3.2.6.2 Traditional applications. Ultrafiltration has often been applied to situations in which it is desirable to recover a valuable component from a wastewater. Successful applications include the recovery of electrophoretic paint, extraction of protein from dairy products, recovery of oil from oil-water emulsions, and recovery of lignosulfonates from spent sulfite liquor in pulping mills (Svarovsky 1979c).

Ultrafiltration has also been used in water reuse applications where the objective is simply to produce water with a quality acceptable for downstream use. Typical applications include sewage treatment, leatherworking industry wastewater treatment, and the treatment of municipal grey wastewater (i.e. laundry and shower wastewater) (Bhattacharyya et al. 1978, cited by Tang, Chu, and Boroughs 1980; Horton 1980). Whittaker (1983) describes a test program in which ultrafiltration was used in conjunction with reverse osmosis to successfully treat waste drilling fluids. The test program demonstrated that high quality effuent could be produced but did not address the issue of how long filter membranes could be expected last under continuous exposure to wastewaters with high sediment concentrations.

3.2.6.3 Advantages and Disadvantages. The primary advantage of an ultrafiltration system applied to disturbed area sediment removal would be its ability to remove virtually all the suspended and colloidal sediment from a given wastewater. A predominance of particles in the one micron size range would not be a hindrance to the system operation. A secondary advantage where space is at a premium would be the very compact nature of ultrafiltration installations.

Offsetting these advantages is the following list of disadvantages:

1. High capital cost;
2. High operating and maintenance costs;
3. Membrane fouling may be a problem;
4. Chemicals may be required for membrane cleaning;

5. Waste concentrate (sludge) may require thickening prior to disposal.

3.2.6.4 Design and operational considerations. In the planning of an ultrafiltration installation, the most important design considerations are (USEPA 1975):

1. Wastewater characteristics,
2. Membrane area,
3. Membrane configuration,
4. Membrane material,
5. Membrane life, and
6. Required pressure.

The required membrane area is determined as a function of the allowable volumetric loading rate in $L/d/m^2$, which is in turn derived from a consideration of the wastewater characteristics and membrane configuration. The volumetric loading rate or 'membrane flux' tends to decrease with time due to surface fouling. To achieve a more stable flux, ultrafiltration processes are normally operated at recirculating feed/concentrate velocities of between 1 m/s and 3 m/s, which are sufficient to scour accumulated solids from the membrane surface (USEPA 1975).

The required membrane area is usually established in conjunction with the selection of one of the membrane configurations described previously. The configuration would be chosen on the basis of the fouling characteristics of the wastewater, cleaning requirements, ease of operation, and cost. Table 50 summarizes some of the advantages and disadvantages of the four major membrane configurations.

The life of the membrane selected would depend to a great extent on the sediment concentration in the wastewater. Unfortunately, quantitative relationships for predicting membrane life as a function of influent quality are not available in the literature, and would have to be established on a site-specific basis, preferably using pilot-scale testing equipment. Where problems with membrane fouling

Table 50. Comparison of configurations of commercially available ultrafiltration membrane equipment.[a]

Major Configuration	Advantages	Disadvantages
Tubular	- Least susceptible to plugging; prefiltration usually not required - Membranes easy to replace in most cases - Small replacement area in most cases - Easy to clean - Proven reliability	- High capital cost per unit filtration area - High recirculation pumping energy requirement (assuming flux rates equal to membranes in configurations with smaller passages)
Flat Plate	- Cost of membrane replacement per unit area should be low - Relatively compact	- Plugging, fouling; prefiltration often required - Membrane replacement sometimes awkward; opening stack may adversely affect many individual membranes - Cost sometimes not much lower than tubular

continued...

Table 50. Concluded.

Major Configuration	Advantages	Disadvantages
Hollow Fibre	- Low capital cost per unit membrane area - Relatively easy to detect element leaks	- Difficult to combine adequate physical properties of fibres with good membrane properties
Spiral Wound	- Low capital cost per unit membrane area - Low energy cost - Compact	- Prefiltration often needed to protect against plugging and fouling - Leak detection may be difficult

[a] Adapted from original table in Horton (1977), cited by Horton (1980).

are anticipated, some method of chemical cleaning would probably be required in conjunction with mechanical cleaning, flushing, or backwashing. When the use of strong chemical cleaning agents is indicated, it is important to select a membrane material that will withstand repeated cleanings, and it is usually prudent to pilot test the proposed cleaning program to confirm membrane durability and to estimate operating costs. Two common membrane materials that are often used in the presence of aggressive chemicals and high temperatures are polysulfone and zirconium oxide (Tang, Chu, and Boroughs 1980).

The pressures used to provide the driving force for ultrafiltration normally range from 140 kPa to 700 kPa. These pressures contrast sharply with those used for molecular filtration (reverse osmosis) devices, which typically vary between 2800 kPa and 8300 kPa (Tang, Chu, and Boroughs 1980).

3.2.6.5 Suitability for sediment removal applications in the Alberta foothills and front ranges. Ultrafiltration offers a substantial advantage over most of the other alternatives discussed thus far in that it has no difficulty removing particles smaller than five microns in diameter. The quality of the produced effluent would be very high, and direct discharge to the nearest watercourse would not pose any problems.

The major difficulty associated with ultrafiltration is the fact that it is not normally used for the kind of suspended solids concentrations that would be encountered in this application. These high sediment concentrations would undoubtedly result in relatively high membrane cleaning/replacement costs, and it is conceivable that these increased costs would be prohibitive. Field studies using pilot scale equipment would be necessary to establish membrane service life and maintenance costs.

3.2.7 Cross Flow Microfiltration

3.2.7.1 Description. Cross flow microfiltration is very similar conceptually to the ultrafiltration technology described in the previous section. The major differences lie in the range of particle sizes separated and the influent suspended solids concentrations that can be tolerated. Crossflow microfiltration separates particles in the 0.1 to 10 micron range while ultrafiltration is usually applied to particle sizes between 0.002 and 1 microns. Crossflow microfiltration is also reported to be capable of handling influent suspended solids concentrations as high as 100 000 mg/L (Neptune Microfloc 1983). Ultrafiltration is normally applied to concentrations several orders of magnitude lower than this figure.

The process hardware required for a cross flow microfiltration system is not unlike that typically used for ultrafiltration installations. Microfiltration systems use different membrane materials and generally require higher capacity flow channels. However in both systems, feedwater is passed tangentially across the surface of the membrane and filtration occurs in a direction perpendicular to the feed flow. The build up of separated solids on the filter surface is reduced by the hydrodynamic shear exerted by the cross flow, permitting the movement of suspended matter across the face of the membrane without fouling at the membrane surface. The feed is continuously recirculated in a pressurized closed loop at a flow many times higher than the permeate flow. The flow pattern used in cross-flow microfiltration is shown conceptually in Figure 86.

To date, the most popular membrane configuration for cross flow microfiltration has been the tubular arrangement described previously for ultrafiltration. The tubes are grouped into bundles and enclosed in a polypropylene enclosure to form what is called a module. Modules are then connected in series in numbers sufficient to satisfy the filtration area requirements. A schematic of a typical installation is shown in Figure 87.

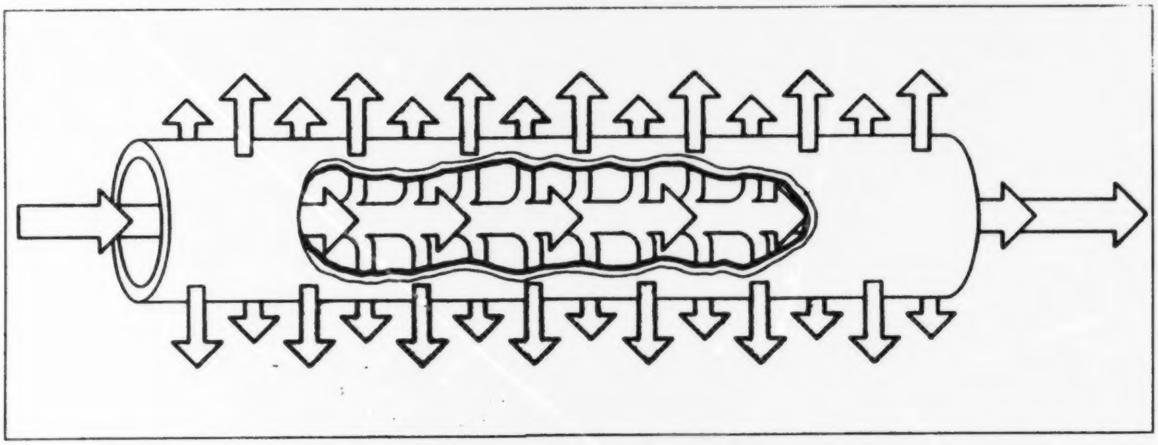


Figure 86. Crossflow microfiltration flow pattern. (Neptune Microfloc 1983)

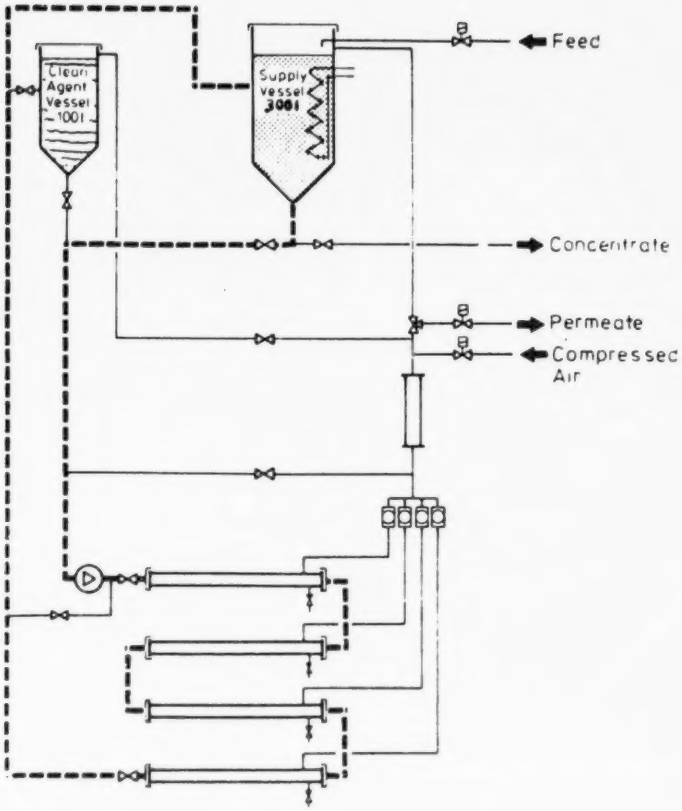

Clean Agent Vessel 100l
Supply vessel 300l
Feed
Concentrate
Permeate
Compressed Air

Figure 87. Schematic representation of a crossflow microfiltration unit. (Schneider and Klein 1982)

The actual membrane required for microfiltration systems differs substantially from those used in ultrafiltration applications. Crossflow microfiltration uses thick-walled, porous plastic tubes that can be extruded from a variety of materials. The tubes are self-supporting and can be manufactured with pore diameters from 0.2 to 0.5 microns depending on the intended application. The actual tube material is selected to be compatible with the nature of the wastewater. Characteristic of the plastics normally used is the polypropylene material described by Schneider and Klein (1982). Figure 88 shows a typical assemblage of tubes enclosed in a module.

3.2.7.2 Traditional applications. Microfiltration has been used for many years in the pharmaceutical and electronics industries for the production of ultra-pure and sterile liquids. However, most of these applications have made use of 'dead-end' microfiltration, in which the feed is not recirculated over the filtration surface. The use of crossflow microfiltration for the treatment of solids-laden inflows is relatively new, and references in the literature to currently operating installations are limited. Neptune Microfloc (1983) reports that the technology has been used in lime softening applications, an example of which is the recently installed potable water treatment plant at Nanton, Alberta.

3.2.7.3 Advantages and disadvantages. Crossflow microfiltration has the potential to offer the following advantages:

1. High effluent quality;
2. Capable of handling fine particle size distributions;
3. High allowable solids loading;
4. Sludge density comparable to that produced by clarifiers (i.e., higher than that normally possible with ultrafiltration);
5. Relatively simple to operate; and
6. Complete installation would be relatively compact.

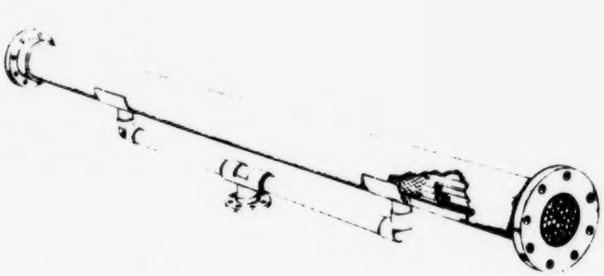


Figure 88. Typical crossflow microfiltration module. (Neptune Microfloc 1983)

A significant disadvantage of microfiltration is likely to be cost. Reliable cost information is scarce, but the physical similarities with ultrafiltration suggest that capital costs could be high. Operating costs would be highly dependent on filtration module cleaning/replacement costs that unfortunately were not clearly defined in the literature. One other drawback of crossflow microfiltration is the fact that it is a relatively new technology. It is essentially untried in many applications, consequently design parameters based on actual operating experience are generally not available.

3.2.7.4 Design and operational considerations. The following parameters must be considered during the design of a crossflow microfiltration facility:

1. Wastewater characteristics,
2. Membrane material,
3. Membrane pore size,
4. Recirculating feed flow velocity,
5. Permeate flux (daily effluent production per unit of filtration area),
6. Backflushing/chemical cleaning routine, and
7. Required pressure.

The membrane material and pore size are selected to be compatible with the characteristics of the wastewater. The treatment of aggressive or corrosive wastewaters requires that the chemical resistivity of the various plastic membranes be examined to ensure that the material selected is not subject to attack by any of the wastewater constituents. The membrane pore size is chosen on the basis of the sediment particle size distribution of the incoming feedwater. For disturbed area sediment removal applications, a relatively coarse pore size in the vicinity of 0.5 microns would probably be selected to reduce fouling and to maximize permeate flux.

The single most important parameter in the design and operation of crossflow microfiltration systems is probably the recirculating feed flow velocity. It is the feed flow velocity that creates the hydrodynamic shear at the membrane surface, which in turn defines the depth of solids accumulation and the permeate flux. In crossflow microfiltration devices, most of the sediment is actually retained by these accumulated solids, so that the definition and maintenance of an optimum solids depth is necessary. Permeate flux has been shown to increase exponentially with shear rate, so that when establishing a recirculating feed flow velocity, a balance has to be drawn between maximizing effluent production (permeate flux) and the maintenance of an accumulated solids depth sufficient for adequate filtration. For most installations, the recirculating feed flow rate is between 3 and 10 times the filtrate flow (Schneider and Klein 1982; Svarovsky 1979c).

The level of permeate flux is one other characteristic that distinguishes crossflow microfiltration from ultrafiltration. Microfiltration systems are capable of flux rates in the order of 40 000 L/d/m^2 to 120 000 L/d/m^2, while ultrafiltration units typically achieve flux rates in the region of 2000 L/d/m^2. The pressures used to produce these microfiltration flux rates range between 140 kPa and 350 kPa (Neptune Microfloc 1983).

In spite of the crossflow characteristics of the recirculating feed, the permeate flux drops with time and eventually reaches a point where backflushing and/or chemical cleaning is required. Figure 89 qualitatively shows the reduction in filtrate flow with time and the effects of backflushing and chemical cleaning. Backflushing is normally accomplished with a one- to two-second shock pressure induced using compressed air in a direction opposite to the permeate flow. The required frequency of backflushing is a function of influent solids concentration and the hardness of the sediment particles. Schneider and Klein (1982) report that hard, crystalline particles are relatively easy to separate and would necessitate comparatively infrequent backwashing. On the other hand, easily deformable particles can cause rapid clogging of the membrane surface, increasing the required frequency of backwashing. It should be noted

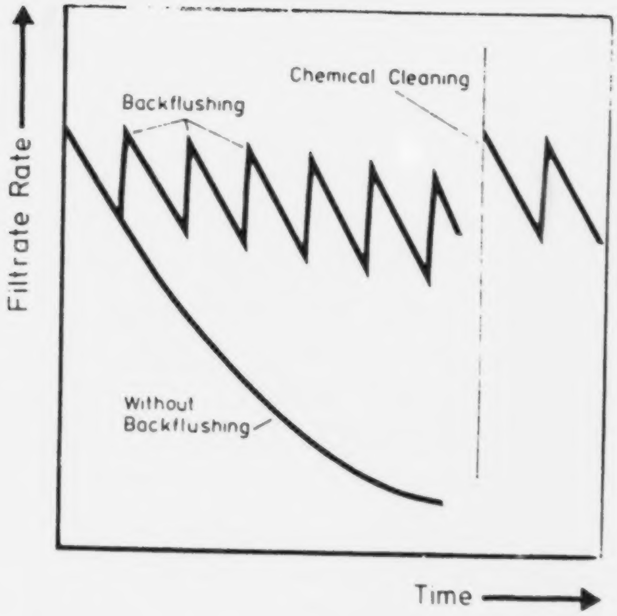


Figure 89. Crossflow microfiltration filtrate rate as a function of time. (Schneider and Klein 1982)

that not all commercial - scale crossflow microfiltration systems include a backflushing cycle. The Neptune Microfloc system, for example, provides only for chemical cleaning, therefore it might not be suitable for applications involving high concentrations of deformable particles.

During the chemical cleaning phase, an acidic or caustic solution is flushed through the filtration tubes to restore the membranes to their original condition. The exact nature of the cleaning solutions used tends to vary slightly between manufacturers and is established as a function of the characteristics of the wastewater to be treated.

3.2.7.5 Suitability for sediment removal applications in the Alberta foothills and front ranges. The available literature suggests that crossflow microfiltration has a strong potential for successful application to disturbed area sediment removal problems. The technology appears to be capable of producing high quality effluent suitable for direct discharge from concentrated influents with very fine particle size distributions.

Cost considerations aside, the major difficulty with crossflow microfiltration is the fact that it is presently an emerging technology. Although all of the alternative technologies discussed thus far suffer from the disadvantage of having rarely ,if ever, been used in this application, crossflow microfiltration has the added disadvantage of having rarely been applied to any full-scale wastewater treatment problem. This means that the process capabilities presented in the literature are subject to a relatively high degree of variability and must be interpreted with caution. The preceding comments notwithstanding, crossflow microfiltration probably represents one of the most promising alternative technologies that could be applied to disturbed area sediment removal problems.

3.2.8 Electrokinetic Methods

3.2.8.1 Description. It has been known for many years that electricity can be used to separate very fine particles from solution and to accelerate the dewatering of sludge. The two physical processes involved in these phenomena are known as electrophoresis and electro-osmosis. Electrophoresis refers to the migration of small, electrically charged particles through a stationary liquid due to an externally applied electrical potential. Electro-osmosis is the movement of liquid through a stationary porous solid as the result of an externally applied electrical potential (Roe 1983).

Electrophoresis can be used to accelerate the settling of well-dispersed, fine-grained clay particles. Clay particles in their dispersed state have a high negative surface charge that, in the presence of a DC electric field, increases their tendency to migrate towards an anode (positive element) placed in solution. Electro-osmosis is used to dewater electrophoretically produced sludge by encouraging the movement of water out of the accumulated solids. The water immediately adjacent to the negatively charged sludge particles contains excess dissolved positive ions that, in effect, give the water a positive charge. These positive ions are repelled by the positive charge carried by the anode on which the sludge is deposited. Water is then carried out of the sludge by the viscous drag of the positive ions as they migrate away from the anode towards the cathode (negative element) (Roe 1983).

The difficulty with electrokinetic methods in the past has been the lack of the hardware necessary for applying the above principles to typical sediment removal applications. One recently developed device that appears to have the potential for disturbed area sediment removal applications is the electrically augmented vacuum filter. This machine, also known as an electrofilter, was jointly developed by Dorr-Oliver Incorporated and the Freeport Kaolin Company for use in the wet wash kaolin industry. The processes that occur in the electrofilter are shown conceptually in Figure 90. The separation process can be summarized as follows:

1. Solids are electrophoretically deposited on the anode in a dense cake;
2. After the water is driven from the deposited cake by electro-osmosis, it migrates towards the cathode;
3. A vacuum is applied at the cathode to draw the wastewater through a filter cloth upon which a thin solids cake is deposited. This solids cake is continuously maintained at a constant thickness and acts as a filter precoat that provides a very high quality effluent; and
4. The anode cake is collected by periodically removing the anode from the bath and harvesting the cake.

Physically, the electrofilter consists of a bath tank in which the anode and cathode elements are suspended. The bath tank supports a number of beams, which in turn support an alternating configuration of anode and cathode elements. A tending device travels down the tank at 10-minute intervals, removing each anode beam in turn and scraping the deposited cake onto a conveyor (Freeman 1982; Dorr-Oliver 1984). A scale model of an electrofilter is shown in Figure 91.

3.2.8.2 Traditional applications. Electrically augmented vacuum filtration is a relatively new technology , consequently examples of proven applications are limited. Apart from the kaolin clay recovery application for which it was developed, the manufacturers suggest that the technology could be applied in the mineral processing industry for the recovery of bentonite clay, ceramics, coal, hectorite clay, taconite, and titanium dioxide (Dorr-Oliver 1984). Unfortunately, no examples were found in the literature of applications in which the recovered solids had no marketable value (i.e., wastewater teatment applications).

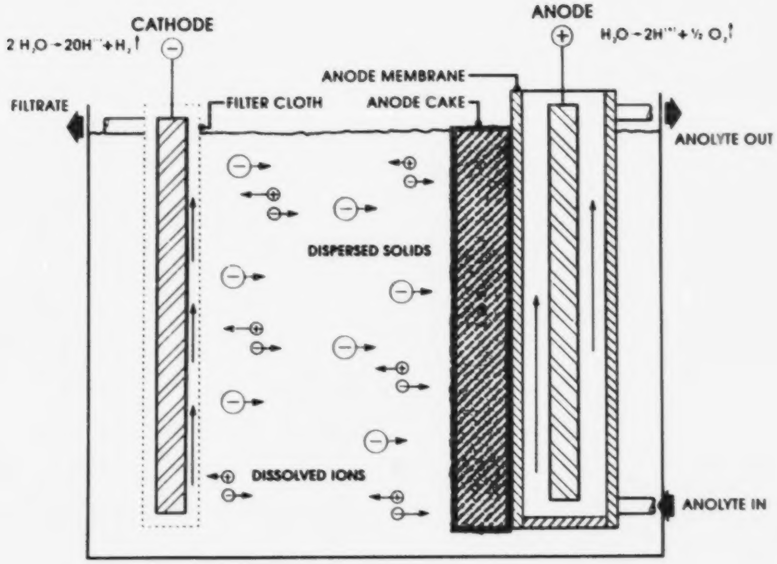

CATHODE
2 H2O → 2OH⁻ + H2↑
ANODE
H2O → 2H⁺ + ½ O2↑
ANODE MEMBRANE
FILTRATE
FILTER CLOTH
ANODE CAKE
ANOLYTE OUT
DISPERSED SOLIDS
DISSOLVED IONS
ANOLYTE IN

Figure 90. The electrofiltration process. (Dorr-Oliver 1984)

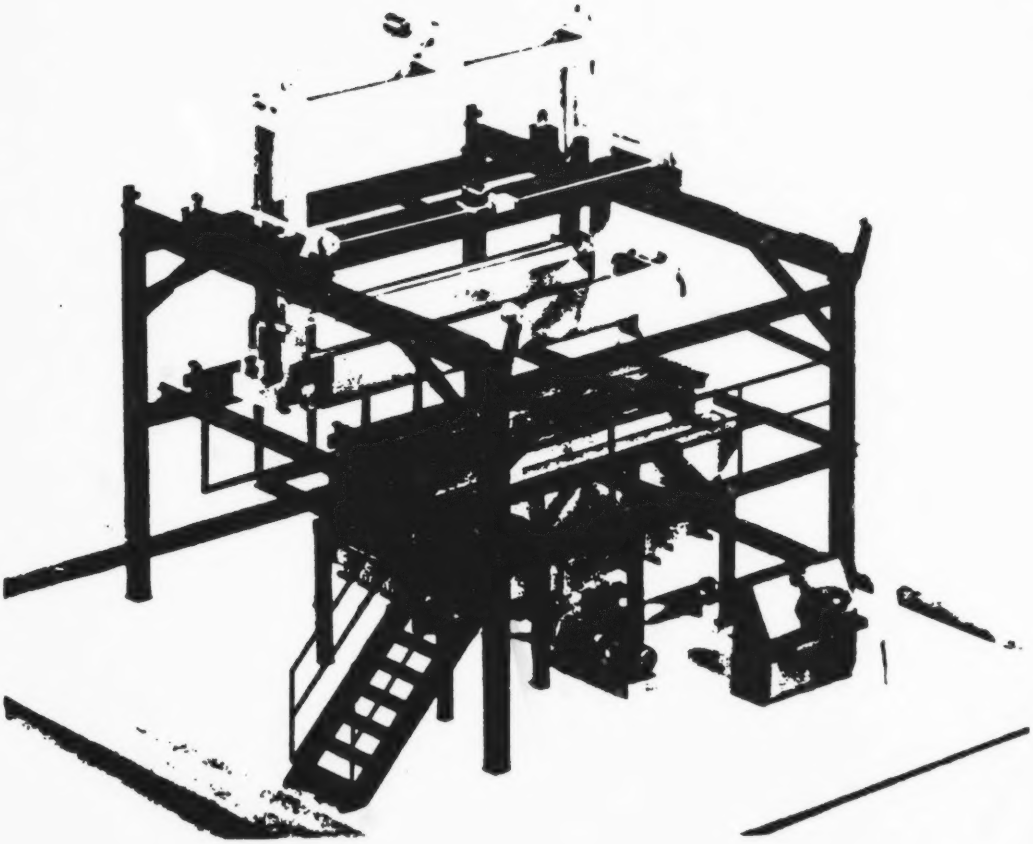


Figure 91. Scale model of the electrofilter. (Freeman 1982)

3.2.8.3 Advantages and disadvantages. Electrofiltration offers the following potential advantages:

1. High effluent quality;
2. Capable of handling very fine particle size distributions;
3. Very high allowable solids loading;
4. High sludge density; and
5. Complete installation would be relatively compact.

Balancing these potential advantages are the following disadvantages:

1. High capital cost;
2. High energy requirement;
3. Incapable of tolerating high concentrations of particles larger than 10 microns in the feedwater;
4. Many require dispersants to deflocculate the incoming sediment if the raw wastewater particles exhibit a high degree of natural aggregation; and
5. Hardware is relatively complex.

3.2.8.4 Design and operational considerations. The following parameters are those that have the strongest influence on the efficiency of an electrofilter (Freeman 1982):

1. Feedwater suspended solids concentration;
2. Feedwater solids particle size distribution;
3. Specific gravity of solids;
4. Zeta potential; and
5. Electrical conductivity.

Electrofiltration can be used to separate and/or dewater solids in feedwaters with solids contents as high as 75% by weight. The distribution of particle sizes in the feedwater is probably as, or more important, than the concentration of incoming solids. Electrofiltration relies on a high concentration of fine particles and therefore, equipment manufacturers suggest that the particle top size be limited to 10 microns. For disturbed area sediment removal applications this would probably necessitate the installation of a sedimentation or filtration facility upstream of the electrofilter (Dorr-Oliver 1984).

Efficient electrofiltration demands that the sediment particles exhibit high zeta potentials. The zeta potential is a measure of the strength of the negative charge on the colloidal particles. The maintenance of a high negative particle charge is necessary for effective electrophoretic migration. For the particles to have a sufficiently high zeta potential, the solution must be highly dispersed. This means that the concentration of positive ions (e.g., sodium, calcium, magnesium) must be kept to a minimum so that the destabilizing influence of Van der Waals' forces and Brownian movement do not promote particle aggregation.

Electrical conductivity is, among other things, a measure of the positive ion concentraiton in a wastewater. High electrical conductivities are indicative of high positive ion concentrations, which in turn are representative of low zeta potentials. In order to limit conductivities, a wastewater should be low in dissolved salts and acids. Dorr-Oliver (1984) suggests that efficient electrofiltration requires electrical conductivities below 5000 microseimens/cm.

Once the preceding parameters have been defined for a particular wastewater, they can be used in conjunction with the proposed physical characteristics of the electrofilter to predict the energy input required for a given solids production. The filter element areas and energy inputs are then varied until an optimum balance is struck between capital (element area) and operating (energy) costs for the desired rate of solids production. A detailed methodology that can be used to complete this exercise is presented in Freeman (1982).

3.2.8.5 Suitability for sediment removal applications in the Alberta foothills and front ranges. In terms of process capabilities and development, electrofiltration is similar to the crossflow microfiltration technology discussed in the previous section. It is capable of producing high quality effluent from concentrated influents with very fine particle size distributions. However, like crossflow microfiltration, electrofiltration is an emerging technology that suffers the disadvantage of having been used in very few commercial-scale applications. Consequently, the complete range of operating capabilities for electrofiltration has yet to be precisely defined.

3.2.9. Cost Considerations

This section examines the relative capital and operating and maintenance costs for the alternative technologies described in previous sections. This evaluation was completed by estimating costs for the treatment of a common wastewater using each of the alternative technologies. The characteristics of the common wastewater, which are summarized in Table 51, were selected to be representative of runoff discharged from disturbed areas in Alberta. The one unusual feature of this wastewater is its relatively fine particle size distribution. This distribution was intentionally biased toward finer gradations since it is for this range of particle sizes that alternatives to sediment removal by ponding are most likely to be sought.

Costs for the chemical treatment, centrifugation, vacuum filtration, and hydrocyclone technologies were based on estimates derived from various references (Gumerman, Culp, and Hansen 1979; Nawrocki and Pietczak 1976) supplemented with in-house cost data. Cost information for the ultrafiltration, crossflow microfiltration, and electrokinetic technologies was provided by equipment suppliers. The results of the cost evaluation are summarized in Table 52. An estimate for a settling pond incorporating chemical feed facilities has been included in the table for comparative purposes. The costs appearing in Table 52 were modified from their original source values to reflect first quarter 1985 Canadian conditions as follows:

Table 51. Alternative technologies cost evaluation and common wastewater characteristics.

1. Average annual flow = 700 L/m
2. Peak annual flow = 2800 L/m
3. Average annual TSS = 400 mg/L
4. Average annual sediment particle size distribution:

Size Range (microns)	TSS (mg/L)	% of Total TSS
0.5 - 1.2	80	20
1.2 - 5	80	20
5 - 8	60	15
8 - 11	60	15
11 - 25	60	15
greater than 25	60	15

Table 52. Alternative technology cost evaluation.[a]

Technology	Capital Cost[e] ($)	Average Annual Polymer Requirement (mg/L)	Annual Operating and Maintenance Cost ($/yr)	Average Produced Sludge Density (% solids by weight)	Annual Sludge Disposal Cost ($/yr)	Total Annual Cost for 3-Year Operation[f] ($/yr)	Total Annual Cost for 10-Year Operation[f] ($/yr)	Total Annual Cost for 15-Year Operation[f] ($/yr)
1. Chemical Treatment Plant	1 900 000	100	230 000	5	16 000	957 000	504 000	442 000
2. Centrifugation[b]	2 400 000	50	310 000	5	16 000	1 224 000	652 000	573 000
3. Vacuum Filtration[c]	900 000	50	180 000	10	9 000	526 000	311 000	282 000
4. Hydrocyclones	150 000	-	30 000	5	16 000	102 000	66 000	61 000
5. Ultrafiltration	2 800 000	-	220 000	4	22 000	1 289 000	623 000	530 000
6. Crossflow Microfiltration	1 700 000	-	120 000	5	16 000	772 000	367 000	311 000
7. Electrokinetic Methods[d]	2 500 000	-	120 000	50	4 000	1 059 000	464 000	382 000
8. Settling Pond	150 000	75	70 000	3	40 000	166 000	130 000	125 000

continued....

Table 52. Concluded.

---

[a] All costs given in first-quarter 1985 Canadian dollars.

[b] Using solid bowl-sedimenting centrifuge.

[c] Using rotary drum vacuum filter.

[d] Using electrically augmented vacuum filter (electrofilter).

[e] Including sludge handling and disposal equipment cost.

[f] Including capital costs annualized using sinking fund depreciation @ i = 6%.

1. Pre-1985 costs were updated using indices presented in the periodicals 'Engineering News Record' and the 'Journal of the Water Pollution Control Federation';
2. American dollars were converted to Canadian currency assuming \$1.00 US = \$0.75 Canadian; and
3. Canadian labour productivity was assumed to be 90% of American labour productivity.

Capital cost estimates include all buildings and ancillary service requirements in addition to an allowance for administration, engineering, interest during construction, and contingency equal to 35% of the total capital cost. Any charges associated with the procurement of land have not been included in the estimates. Operating and maintenance costs include an allowance of 10% for administration and supervision.

The average annual polymer requirements shown in the table are intended to approximate those that would be necessary to provide a degree of flocculation sufficient for the production of acceptable effluent quality. Costs for the polymer dosage noted are included in the annual operating and maintenance figures based on a unit chemical charge of \$2.00/ kg.

The annual sludge disposal costs shown in Table 52 represent the annual operating and maintenance charges for transporting sludge of the density noted to its final disposal location. Capital costs for sludge handling equipment are included in the total capital cost figure presented for each technology. The sludge disposal estimates are based on a unit cost of \$5.00 per cubic metre of annual sludge production. This figure is consistent with in-house data relating to the cost of sludge disposal via truck to an ultimate disposal location approximately 4 km from the site of sludge production.

To illustrate the variation in total costs between alternatives, the final section of Table 52 includes estimates of total system costs for 3-, 10-, and 15-year operations. These figures represent the total annual cost, including annualized capital charges and sludge disposal costs, for each alternative technology. Capital costs were converted to annual costs using capital recovery factors of 37.4%, 13.6%, and 10.3% for the 3-, 10-, and 15-year operations respectively. These factors were calculated using sinking fund depreciation with a real rate of return of 6%. The 6% rate of return did not include an inflation component and was intended to reflect the real cost of capital to the operator of the technology. Taxes and insurance costs were not included and it was assumed that the equipment involved would not have any salvage value at the termination of operations. This final assumption is probably somewhat unrealistic for short-term (e.g., 3-year) operations. It is likely that unless dismantling and/or transportation costs are very high, the equipment could be moved and reused at another location.

It must be recognized that the cost figures shown in Table 52 are order-of-magnitude estimates only. Few of the technologies shown have been applied with any frequency to the sort of disturbed area sediment removal problem in question. Cost information and appropriate equipment design criteria were therefore scarce, requiring that numerous simplifying assumptions be made for the preparation of the estimates shown. The figures presented should be interpreted only as best estimates of relative alternative costs and should not be modified or extrapolated to suit particular sites in an effort to predict actual costs for a certain technology. Reliable cost estimates for individual applications would have to be based on site-specific bench- or pilot-scale field data.

The relative costs shown in Table 52 are basically in line with what one would intuitively expect regarding the cost differential between the generally complicated alternative technologies and the traditional settling pond technology. With the notable exception of hydrocyclones, all of the alternative technologies are substantially more expensive than settling ponds. The conclusion that can be drawn from this is that unless there is a very large proportion of fines in the feedwater, or the space available for impoundment construction is extremely limited, settling ponds will probably represent the most cost-effective solution for most sediment removal applications. When settling pond construction is either not feasible, or is incapable of providing the desired effluent quality, vacuum filtration, crossflow microfiltration, and electrokinetic separation appear to be the most cost-effective alternatives.

The simplicity of hydrocyclones does make them attractive economically; however, the limited capability of these devices to retain particles smaller than 10 microns in diameter reduces their suitability for many applications. Unfortunately, the severe velocity gradients generated by the flow pattern in a hydrocyclone preclude the use of polymer flocculants. The velocity gradients are so great that any flocculated material would quickly be broken up into components of non-separable sizes. Nevertheless, the low cost of hydrocyclones could make them an attractive alternative for very confined areas discharging runoffs with relatively coarse-grained sediment loads. Hydrocyclones could also serve as a reasonably inexpensive prethickener for other alternative technologies.

4. DISCUSSION

4.1 SETTLING POND DESIGN

4.1.1 Sedimentation Models

The various sedimentation models presented in this document vary greatly in sophistication and in their ability to accurately predict settling pond performance. Generally speaking, and assuming well defined input data, the more sophisticated models (e.g., DEPOSITS, CSTRS, and BASIN) provide the most accurate results. The problem that arises when using these models is that meaningful input data is often not readily available. The relatively large number of inputs that sophisticated models use and the sensitivity of the results to the quality of those inputs, mean that the use of these models with poorly defined input data may produce results that are no better, or only marginally better, than those provided by one of the less complicated sedimentation models. An illustration of the sensitivity of sophisticated model outputs (in this case DEPOSITS used as a subroutine of the SEDIMOT II watershed model) to relatively modest variations in some of the more significant input parameters is presented in Table 53. The preceding comments notwithstanding, the considered and experienced use of sophisticated computer models can be used to provide at least educated insights into the probable range of performance for a proposed settling pond. An indirect advantage of these models is that the necessity to gather relatively large quantities of input data forces the user to be more aware of those parameters that have an influence on pond performance. This would tend to improve the quality of input data used over that typically gathered for some of the less sophisticated design models.

The problem of input data definition is particularly acute when dealing with a proposed land disturbance for a virgin watershed. In this case, regardless of the pond design methodology used, field studies should be conducted to establish the characteristics of the watershed and its overburden. Particular attention should be paid to

Table 53. Sensitivity of peak effluent concentrations predicted by the DEPOSITS[a] model to variations in input parameters.

Parameter	% Change in Peak Effluent Concentration	
	+10% Parameter Change	-10% Parameter Change
Runoff curve number (CN)	31	-44
Watershed runoff time of concentration (Tc)	-2	1
Soil erodibility coefficient (K)	13	-7
Slope length (L)	5	-5
Percent grade for slope length (% Slope)	13	-13
Control practice factor (CP)	10	-10
Specific gravity of eroded sediment (SG)	-25	36

[a] Used as a subroutine of the SEDIMOT II watershed model.

the estimation of meaningful eroded sediment particle size distributions. This is the single most important parameter influencing settling pond performance and cost. It is important to note that reliable particle size distribution estimates cannot be derived simply from the size distribution of the in-situ overburden. Procedures such as the one described in Section 2.11.3 should be used to account for the influence of rainfall energy and soil erodibility on disturbed soils. When defining input parameters for virgin watersheds, it is also important to carefully evaluate the nature of the proposed land disturbing activity. If non-self-draining pits are proposed, estimates of the quantity and quality characteristics of the pumped flow should be made. When defining these characteristics, consideration should be given to the extent and nature of any partings that may be encountered if the land disturbing activity involves the extraction of a mineral resource.

If the land disturbing activity is conducted over a long period of time and involves the construction of a number of impoundments at various times, consideration should be given to establishing a data gathering program after the first settling pond is put into operation. Such a program could provide the well-defined input data that would be necessary to cost-effectively design subsequent sediment removal facilities with an assurance that the appropriate regulatory requirements would be satisfied. Some of the more relevant parameters that could be included in such a data-gathering program are:

1. Watershed storm event summaries, including rainfall depth, intensity, and duration;
2. Pond influent flow;
3. Pond influent TSS;
4. Pond influent particle size distribution;
5. Specific gravity of sediment particles;
6. Pond effluent TSS;
7. Pond effluent particle size distribution;
8. Sludge depth;
9. Sludge density;

10. Polymer dosages (if any); and
11. Relationship between polymer dosage and influent particle size distribution (if applicable).

When gathering this data for uncontrolled flow conditions, it would be necessary to correlate the characteristics of pond influents (i.e., flow, TSS, and particle size distribution) to the nature of the storm events causing them, so that the actual inlet hydrograph and sedimentgraph for the design storm event could be estimated. If a data-gathering program similar to the one described above is implemented for a long-term operation, the use of sophisticated computer models for the design of second and subsequent ponds has the potential to produce relatively cost-effective designs that provide greater assurance that the regulatory requirements will be met.

4.1.2 Current Alberta Practice

As noted in Section 2.9.1.2.4, a very common practice in Alberta is to design settling ponds to remove all particles larger than a particular 'cut size'. It is also commonly assumed that if significant concentrations of sediments smaller than this cut size are encountered, polymer flocculants can be used to provide the particle size distribution necessary for adequate pond performance. The problem with this approach is that an estimate of the actual incoming particle size distribution is not a required input parameter. This tends to reduce the effort expended in estimating particle size distributions, which in turn can result in the selection of a cut size that is poorly correlated to actual site conditions. The selection of an inappropriate cut size can lead to a poorly optimized balance between capital (pond) costs and operating (polymer) costs. If the cut size selected is too large, the pond may be undersized and require high ongoing polymer costs to maintain acceptable pond performance. Conversely, if the cut size is too small, the high capital charges associated with the overly large pond size predicted may never be recovered by reduced polymer charges during the service life of the pond. The potential for these additional charges to be incurred could be reduced by making a concerted effort to derive a reasonable estimate of the incoming sediment particle size distribution from site-specific field data.

4.1.3 Designing Settling Ponds for Alberta Conditions

There are at least two particulars of the Alberta condition that should be considered in settling pond design. The first and most obvious is the relatively harsh winter weather experienced in most parts of the province. The second is that a high proportion of land disturbing activities in Alberta discharge at least a portion of their wastewater as controlled (pumped) rather than uncontrolled (storm) flow. These factors can be considered in pond design by evaluating pond performance under the following conditions:

1. Summer conditions with the 1 in 10 year, 24-hour uncontrolled storm flow using the sediment concentration and particle size distribution associated with that storm (this would only apply if a significant volume of pond inflow were uncontrolled);
2. Summer conditions with both the maximum and the most protracted controlled (pumped) flows using the sediment concentrations and particle size distributions associated with those flows (this would only apply if a significant volume of pond inflow were controlled; and
3. Winter or spring conditions with the maximum controlled (pumped) winter or spring flow in conjunction with a surface ice cover and reduced water temperatures and the sediment concentrations and particle size distributions associated with the controlled flow in question.

In many cases, some of the above conditions could be identified as being unlikely to be critical simply by observing the relative magnitudes of storm and pumped flows and any differences in particle size distributions between storm and pumped flows. For example, if the controlled flow particle size distribution is the same or coarser than the storm flow distribution, it is unlikely that the controlled flow condition would be more critical than the storm flow condition unless storm flows represent only a small percentage of the total pond inflow.

For those ponds whose inflow is predominately uncontrolled, the summer condition noted above will almost always be more critical than spring conditions during which snowmelt and rainfall may be discharged simultaneously to an impoundment with a surface ice cover. This is because the 1 in 10 year, 24 hour rainfall normally occurs later in the year than snow and/or ice is likely to be present. Table 54 lists the maximum 24 hour rainfall for a number of Alberta locations and shows that for the noted areas, the critical 24 hour storm never occurs earlier than June and is normally of a considerably greater magnitude than the greatest April or May storms.

4.2 ALTERNATIVES TO SEDIMENT REMOVAL BY PONDING

4.2.1 Augmentative Technologies

It is clear from the section on augmentative technologies that these are not stand alone techniques. The application of augmentative methods alone will not meet an effluent TSS or settleable solids standard. The methods must be used in conjunction with some other treatment techniques to achieve the desired sediment removal goals.

The most important area of application for the augmentative technologies is in minimizing erosion and transportation of sediment. In this respect the techniques should form part of a well planned sediment control strategy. Implementation of some of these techniques will also have long-term positive benefits when the site is ready for reclamation.

4.2.2 Alternative Technologies

The comments presented in previous sections on the various alternative technologies are summarized in Table 55. The table

Table 54. Greatest rainfall in 24 hours at selected Alberta locations.[a]

Location	Maximum 24-Hour Rainfall (mm)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Hinton	3.8	2.5	2.3	11.4	34.3	70.4	49.5	54.1	29.7	17.3	3.6	5.1
years[b]	12	13	15	10	12	11	15	15	15	15	14	14
Jasper	18.8	8.1	15.7	14.8	70.4	36.8	86.6	107.7	32.8	29.7	16.3	19.6
years[b]	49	48	49	50	50	49	49	48	47	49	49	48
Grande Cache	8.9	3.0	5.0	11.9	41.0	88.9	40.0	44.5	29.5	11.4	21.8	24.9
years[b]	6	8	7	9	8	9	9	9	8	8	8	7
Edson	7.1	5.3	4.6	16.5	36.6	33.0	68.6	58.7	30.0	13.0	3.3	5.3
years[b]	10	11	11	11	10	10	10	10	10	10	10	10
Embarras	0.3	1.5	2.3	12.7	21.1	42.2	81.8	50.8	37.6	26.7	7.6	5.1
years[b]	19	20	20	19	20	20	20	20	19	19	18	19
Brazeau	-	-	-	15.2	54.1	67.3	80.8	50.3	43.4	19.8	-	-
years[b]	0	0	0	5	34	39	39	37	35	20	-	0

[a] Source: Canada Environment (1982).

[b] Years of record for observed precipitation event.

Table 55. Summary of alternative technologies.

Technology	Potential for Producing Acceptable Effluent from Influents with Fine Particle Size Distributions[a,b]	Ability to Tolerate High Suspended Solids Concentrations while Maintaining Consistent Effluent Quality[a,c,d]	Required Level of Operating and Maintenance Input and Expertise[a]	Space Requirements[a]	Estimated Total Annual Cost for 10-Year Operation[e] ($/yr)
1. Chemical Treatment Plant	H	L	H	H	504 000
2. Centrifugation	M	VH	H	M	652 000
3. Vacuum Filtration	M	VH	H	M	311 000
4. Hydrocyclones	L	VH	L	M	66 000
5. Ultrafiltration	VH	L	M	L	623 000
6. Crossflow Microfiltration	VH	H	M	L	367 000

continued...

Table 55. Concluded.

Technology	Potential for Producing Acceptable Effluent from Influents with Fine Particle Size Distributions[a,b]	Ability to Tolerate High Suspended Solids Concentrations while Maintaining Consistent Effluent Quality[a,c,d]	Required Level of Operating and Maintenance Input and Expertise[a]	Space Requirements[a]	Estimated Total Annual Cost for 10-Year Operation[e] ($/yr)
7. Electrokinetic Methods	VH	VH	H	M	464 000
8. Settling Pond	L	M	VL	VH	130 000

[a] Rated qualitatively on the following scale:
VL = Very Low, L = Low, M = Moderate, H = High, VH = Very High

[b] Acceptable effluent quality is assumed to be one with a suspended solids concentration at or below 50 mg/L.

[c] 'High' TSS levels are assumed to be those above approximately 10 000 mg/L.

[d] Effluent quality maintained is not necessarily within compliance limits.

[e] Operating and maintenance charges plus capital costs annualized using sinking fund depreciation @ i = 6% and n = 10 years; all costs expressed in first quarter 1985 Canadian dollars. Costs are based on wastewater flow and characteristics noted in Table 51.

presents a qualitative rating of each alternative with respect to a number of parameters and the estimated total annual cost for the alternative based on the wastewater characteristics presented in Section 3.2.9. A settling pond incorporating polymer flocculants as an augmentative technology has been included for comparative purposes.

It should be noted that none of the alternatives summarized in Table 55 are likely to ever be used as a totally independent sediment removal technology. Since these technologies do not incorporate any significant storage volume internally, their capacity to dampen short term peak flows is very limited and consequently, each system would have to be sized on the basis of peak incoming flow. During 1 in 10 year storm conditions these peak inflows can be significantly higher than the average annual flow. If treatment systems had to be designed to handle these storm flows, capital costs would be very high. Unless costs for impoundment construction are unusually high, it is likely that it would be more cost effective to store these peak storm flows in upstream lagoons rather than provide the equipment capacity needed to treat them directly. The probable use of this upstream storage was recognized implicitly during the preparation of the cost estimates appearing in Table 55. Although the maximum flow used to derive these estimates was higher than the average annual flow, it was still considerably less than what one could reasonably expect for the 1 in 10 year peak storm flow. The optimum combination of upstream storage and treatment facility capacity would have to be determined individually for each site and for each alternative technology and would depend heavily on the ratio of equipment cost to lagoon storage cost for the technology under consideration.

A secondary advantage of upstream storage is that it would provide a measure of pretreatment for the runoff. The coarsest sediment particles would be removed in the storage lagoon ahead of the treatment facility. This would be a significant advantage for those alternative technologies (e.g. ultrafiltration, electrokinetic methods) that cannot tolerate high proportions of large particles in the feed stream.

One issue that should be considered in the formulation of a sediment control strategy is the fact that the most cost-effective sediment removal solution for a particular location may involve the use of a combination of the alternatives summarized in Table 55. Some of these technologies are most effective over a narrower range of influent characteristics than is normally encountered in runoff from disturbed areas. Therefore, in some situations, it may be more economical to use two or even three technologies, each operating over the range of influent characteristics for which its efficiency is maximized. For example, hydrocyclones have some difficulty removing particles smaller than 10 microns, while electrofilters cannot tolerate high proportions of particles larger than 10 microns. A system incorporating hydrocyclones upstream of electrofilters could then be designed to maximize the efficiency of both technologies. Whether or not this complementary use of alternatives would be less costly than a third independent technology would have to be evaluated on a site-specific basis.

5. REFERENCES CITED

Alberta Environment. 1977. Alberta surface water quality objectives. Edmonton, 17 pp.

Alberta Environment. 1978. Alberta coal mining wastewater effluent guidelines. Edmonton, 24 pp.

Alberta Environment. 1979. Dam and canal safety guidelines. Edmonton. 8 pp.

Albrecht, S.C. and B.J. Barfield. 1981. Use of a vegetative filter zone to control fine grained sediments from surface mines. EPA 600/7-81-117, Industrial Environmental Research, US Environmental Protection Agency, Cincinnati, OH.

American Water Works Association. 1969. Water treatment plant design. American Water Works Association. New York, N.Y.

Ballew, H.W. 1978. Basics of filtration and separation. Nuclepore Corporation. CA.

Barfield, B.J. and J.C. Hayes. 1980. Modelling sediment filtration by vegetative filters. ASCE Watershed Management Symposium, Boise, ID.

Barfield, B.J., J.C. Hayes, and R.I. Barnhisel. 1978. The use of grass filters for sediment control in strip mine drainage. IMMR 39-RR4-78, Institute for Mining and Minerals Research, Lexington, KY.

Barfield, B.J., R.C. Warner, and C.T. Haan. 1981. Applied hydrology and sedimentology for disturbed areas. Oklahoma Technical Press, Stillwater, OK.

Beasley, D.B. and L.G. Huggins. 1982. Answers-Users Manual. Report EPA-905/9-82-001. U.S. Environmental Protection Agency. Region V. Chicago, IL.

Bird, R.B., W.E. Stewart and E.N. Lightfoot. 1966. Transport phenomena. John Wiley and Sons. New York, NY.

Brune, G.M. 1953. Trap efficiency of reservoirs. Transactions of the American Geophysical Union. Vol. 34.

Camp, T.R. 1944. Discussion of turbulence and sedimentation. Discussion of Dobbin's ASCE Paper 2218. Transactions ASCE, pp. 660-662.

Camp, T.R. 1945. Sedimentation and the design of settling tanks. Transactions ASCE Paper No. 2285, pp. 895-946.

Canada Enviroment. 1982. Canadian Climate Normals. Volume 3: Precipitation, 1951-1980. Downsview, Ontario. 602 pp.

Chen, C.N. 1974. Evaluation and control of soil erosion in urbanizing watersheds. Proceedings 1974 National Symposium on Urban Rainfall and Runoff and Sediment Control. UK BU106, College of Engineering, University of Kentucky, Lexington, KY.

Chen, Y.N. 1975. Design of sediment retention basins. Proceedings National Symposium on Urban Hydrology and Sediment Control. Lexington, Kentucky.

Churchill, M.A. 1948. Discussion of analysis and use of reservoir sedimentation data by L.G. Gottschalk. Proceedings Inter-Agency Sedimentation Conference. U.S. Department of the Interior.

Clark, J.W., W. Viessman, Jr., and M.J. Hammer. 1977. Water supply and pollution control, 3rd edition. IEP. New York, NY.

Clark, R.A., J.H. Ferziger and W.C. Reynolds. 1979. Evaluation of subgrid-scale models using an accurate simulated turbulent flow.

Culp, R.L., G.M. Wesner, and G.L. Culp. 1978. Handbook of advanced wastewater treatment, 2nd edition. Van Nostrand Reinhold Company. New York, N.Y.

Currie, I.G. 1974. Fundamental mechanics of fluids. McGraw-Hill Inc., New York, NY.

Curtis, D.C. and R.H. Cuen. 1977. Design efficiency of stormwater detention basins. Journal of the Water Resources Planning and Management Division. Proc. ASCE Vol. 103, pp. 125-140.

Dendy, R.D. 1974. Sediment trap efficiency of small reservoirs. Transactions ASAE. Vol. 17, pp. 898-901, 908.

Dobbins, W.E. 1944. Effect of turbulence on sedimentation. Transactions ASCE. Paper No. 2218, pp. 629-655.

Dorr-Oliver Inc. 1984. The Dorr-Oliver electrofilter. Bulletin EF-1. Don-Oliver Inc., Stamford, CT.

Einstein, H.A. 1950. The bed-load function for sediment transportation in open channel flows. U.S. Department of Agriculture. Technical Bulletin No. 126, pp. 1-78.

Ettinger, C.E. and J.E. Lichty. 1979. Evaluation of performance capability of surface mine sedimentation ponds. U.S.EPA Report EPA-440/1-79/200, U.S. Environmental Protection Agency; Cincinnati, Ohio.

Evangelou, V.P. and H.J. Grove. 1981. Soil-water: How they interact and how their interactions can be used to manage sediment ponds. Reference material for the 1981 National Symposium on Surface Mining Hydrology, Sedimentology and Reclamation, University of Kentucky, Lexington, Kentucky.

Evangelou, V.P., H.V. Grove and R.I. Barnhisel. 1984. Identification of soil-water chemical parameters for the prediction and treatment of suspended solids in surface water reservoirs of coal mine lands. Research report, Water Resources Research Institute, University of Kentucky, Lexington, Kentucky.

Evangelou, V.P., R. Fowlings, J.D. Crutchfield and E.A. Shannon. 1981. A simple chemo-mathematical model as a tool in managing surface mine sediment ponds. Proceedings of the 1981 Symposium on Surface Mining Hydrology, Sedimentology, and Reclamation, University of Kentucky, Lexington, Kentucky.

Ford, D.E. and H.G. Stefan. 1980. Thermal predictions using integral energy method. Proceedings of the ASCE, 106 (HY1):39-55.

Foster, G.R. 1982. Modelling the erosion process. Chapter 8 in Haan, Johnson and Brakensiek, Ed. Hydrologic Modelling of Small Watersheds Monograph No. 5, ASAE, St. Joseph, MI.

Freeman, M.P. 1982. Vacuum electrofiltration. Chemical Engineering Progress. August 1982. pp. 74-79.

Gallegos, C.L., F.R. Schiebe, and W. Stefan. 1983. Modelling water movement and thermal structure in agricultural impoundments. Proceedings Natural Resources Modelling Symposium. Fingree Park. Co.

Gibson, M.M., W.P. Jones, J.J. McGuirk and J.H. Whitehead. 1981. Turbulence models for computational fluid dynamics. Lecture notes, Pennsylvania State University short course, University Park, PA.

Graf, W.H. 1971. Hydraulics of sediment transport. McGraw-Hill, Inc. New York, N.Y.

Graver Water. 1982. Reactivator clarifier. Bulletin WC-103E. Graver Water. Union, NV.

Griffin, M.L., B.J. Barfield and R.C. Warner. 1983. Model studies of the dead storage within sediment ponds. Proceedings of the 1983 Symposium on Surface Mining Hydrology, Sedimentology and Reclamation. UK BU 133, College of Engineering, University of Kentucky, Lexington, Kentucky.

Gumerman, R.C., R.L. Culp, and S.P. Hansen. 1979. Estimating water treatment costs - volume 2: cost curves applicable to 1 to 200 mgd treatment plants. Report EPA-600/2-79-162b. U.S. Environmental Protection Agency. Cincinnati, OH.

Haan, C.T. and B.J. Barfield. 1978. Hydrology and sedimentology of surface mined lands. Lexington, Kentucky: Institute for Mining and Minerals Research. 286 pp.

Hayes, J.C., B.J. Barfield, and R.I. Barnhisel. 1979a. Filtration of sediment by simulated vegetation II: Unsteady flow with nonhomogeneous sediment. Trans. ASAE, Vol. 22(5):1063-1067.

Hayes, J.C., B.J. Barfield, and R.I. Barnhisel. 1979b. Evaluation of vegetal filtration for reducing sediment in surface mine runoff. Symposium on Surface Mine Hydrology, Sedimentology, and Reclamation, University of Kentucky, Lexington, KY.

Hayes, J.C., B.J. Barfield, and R.I. Barnhisel. 1982. The use of grass filters for sediment control in strip mine drainage. Vol. III. Empirical verification of procedures using real grasses. Report IMMR 82/070, Institute of Mining and Minerals Research, University of Kentucky, Lexington, KY.

Hayes, J.C., B.J. Barfield, and R.I. Barnhisel. 1984. Performance of grass filters under laboratory and field conditions. Trans. ASAE 27(5):1321-1331.

Heineman, H.G. 1978. U.S. progress in reservoir trap efficiency research. Tenth International Conference on Sedimentology. Jerusalem, Israel.

Hill, R.D. 1976. Sedimentation ponds - a critical review. Proceedings Sixth Symposium on Coal Mine Drainage Research. Louisville, Kentucky.

Hirschi, M.C. and R.C. Warner. 1983. Modelling the trap efficiency of check dams. ASAE, St. Joseph, MI.

Horton, B.S. 1980. Improvements needed in ultrafiltration to increase the recycle of water. Contract No. 14-34-0001-8535. U.S. Department of the Interior. Washington, D.C.

Horton, R.E. 1940. Approach toward a physical interpretation of infiltration capacity. Soil Science Society of American Proceedings. No. 5 pp. 339-417.

Jenkins, C.R. and J.F. Pankanin. 1979. Modification of sediment control ponds. The 34th Industrial Waste Conference. Purdue University, Lafayette, Indiana.

Kathuria, D.V., M.A. Nawrocki and B.C. Becker. 1976. Effectiveness of surface mine sediment ponds. U.S.EPA Report No. EPA-600/2-76-117. U.S. Environmental Protection Agency; Cincinnati, Ohio.

Kirpich, P.Z. 1940. Time of concentration of small agricultural watersheds. Civil Engineering, Vol. 10, No. 6.

Knisel, W.G., Editor. 1980. CREAMS: A field-scale model for chemical, runoff, and erosion from agricultural management systems. USA Conservation Research Report No. 26.

Launder, B.E. and D.B. Spalding. 1972. Mathematical models of turbulence. Academic Press Inc., New York, NY.

Linsley, R.K., Jr., M.A. Kohler and J.L.H. Paulhus. 1975. Hydrology for engineers, 2nd edition. McGraw-Hill, Inc. 482 pp.

Metcalf and Eddy, Inc. 1979. Wastewater engineering, treatment disposal and reuse. McGraw-Hill, Inc. New York, N.Y.

National Car Rental. Undated. Vaughan lagoon pumper. St. Louis Park, MN.

Nawrocki, M. and J. Pietczak. 1976. Methods to control fine-grained sediments resulting from construction activity. Report EPA 440/9-76-026. U.S. Environmental Protection Agency. Washington, D.C.

Neptune Microfloc. 1983. Hydroperm crossflow microfiltration. Bulletin KS11-15. Neptune Microfloc Inc., Corvallis, OR.

Onstad, C.A. and G.R. Foster. 1975. Erosion modelling on a watershed. Transactions of the ASAE 18(2):288-292.

Peterson, E.B. and N.M. Peterson. 1977. Revegetation information applicable to mining sites in Northern Canada. Environmental Studies No. 3. Department of Indian and Northern Affairs. QS-8144-000-EE-A1. Ottawa. 405 pp.

Philip, J.R. 1954. An infiltration equation with physical significance. Soil Science. No. 77.

Poe, M.W., R.P. Betson and R. Singh. 1983. Can sediment ponds meet effluent limitations. Proceedings of the Symposium on Surface Mining, Hydrology, Sedimentology, and Reclamation. 1983; Publication UKY BU 133. University of Kentucky, Lexington, Kentucky.

Rausch, D.L. and H.G. Heinemann. 1975. Controlling reservoir trap efficiency. Transactions ASAE Vol. ?, pp. 1105-1108, 1113.

Reynolds, W.C. 1976. Computation of turbulent flows. Annual Review of Fluid Mechanics. Vol. 8, pp. 103-208.

Roe, C.H. 1983. Review of disliming methods and equipment. Information Circular 8972. U.S. Bureau of Mines.

Ross, B.B., V.O. Shanholtz and D.N. Contractor. 1980. A spatially responsive hydrologic model to predict erosion and sediment transport. Water Resources Bulletin 16(3):538-545.

Rothwell, R.L. 1978. Watershed management guidelines for logging and road construction in Alberta. Fish. Environ. Can., Can. For. Serv., North. For. Res. Cent., Edmonton, Alberta. Inf. Rep. NOR-X-208.

Schiebe, F.R. 1983. A concept for modelling deposited sediment distribution in reservoirs. Proceedings Natural Resources Modelling Symposium. Fingree Park. Co.

Schlichting, H. 1979. Boundary-layer theory. McGraw-Hill Inc. New York, NY.

Schneider, K. and W. Klein. 1982. The concentration of suspensions by means of crossflow microfiltration. Desalination 41(1982) 263-275.

Schroeder, P.R. and F.D. Shields, Jr. 1983. Chemical clarification of dredged material. Proceedings of the ASCE. EE109(2): 414-427.

Simons, D.B. and F. Senturk. 1977. Sediment transport technology. Water Resources Publications. Fort Collins, Co.

Simons, D.B., R.M. Li, G.H. Cotton, R.M. Summer and R.A. Mussetter. 1983. Design of sediment control measures for small areas in surface coal mining. Simons, Li and Associates Report prepared for Office of Surface Mining, Washington, D.C.

Singley, J.E. 1972. Theory and Mechanism of Polyelectrolytes as Coagulant Aids. AWWA Seminar Proceedings on Polyelectrolytes - Aids to Better Water Qualtiy - AWWA Conference, June 4, 1972. No. 20121.

Smith, R.E. 1981. A kinematic model for surface mine sediment yield. Transactions of the ASAE, 24(6):1508-1514.

Stefan, H.G., S. Dhanothanan and F.R. Schiebe. 1984. Temperature/sediment model for a shallow lake. Proceedings of the ASCE, 108 (EE4):750-765.

Stewart, B.A., D.A. Woolhiser, W.H. Wischmeier, J.H. Caro, and M.H. Frere. 1975. Control of water pollution for croplands. EPA-600/2-75-026. Environmental Protection Agency, Washington, D.C.

Svarovsky, L. 1979a. Advances in solids-liquids separations I: filtration and allied operations. Chemical Enginering. Vol. 86 (14) pp. 62-76.

Svarovsky, L. 1979b. Advances in solids-liquids separations II: sedimentation, centrifugation and flotation. Chemical Enginering. Vol. 86 (15) pp. 93-105.

Svarovsky, L. 1979c. Advances in solids-liquids separations III: recent alternatives that boost recoveries or cut costs. Chemical Enginering. Vol. 86 (16) pp. 69-78.

Tajek, J. and W.W. Pettapiece. 1984. Use of Universal Soil Loss Equation in Alberta, presented at the 1984 meeting of the Canadian Society of Soil Science, Banff, Alberta, Canada. 11 pp.

Tang, T.L.D.T., T.J. Chu, and R.D. Buroughs. 1980. Application of membrane technology to power generation wastewaters. Report EPA-600/7-80-063. U.S. Environmental Protection Agency. Washington, D.C.

Tapp, J.S., B.J. Barfield and M.L. Griffin. 1981. Predicting suspended solids removal in pilot scale sediment ponds utilizing chemical flocculation. Technical report of Institute for Mining and Minerals Research; University of Kentucky, Lexington, Kentucky.

Taylor, Gregory S. and Charles R. Jenkins. 1982. Low cost modification of Sediment Control Ponds. 1982 Symposium Surface Mining Hydrology, Sedimentology and Reclamation, pp. 671-680. Lexington, Kentucky.

Tennekes, H. and J.L. Lumley. 1972. A first course in turbulence. MIT Press, Cambridge, Massachusetts.

Thronson, R. 1976. Methods to control fine-grained sediments resulting from construction activity. Report EPA 440/9-76-026. U.S. Environmental Protection Agency, Washington, D.C.

Tollner, E.W., B.J. Barfield, C. Vachirakornwatana, and C.T. Haan. 1977. Sediment deposition patterns in simulated grass filters. Trans. ASAE, Vol. 20(5):940-944.

Tollner, E.W., B.J. Barfield, C.T. Haan, and T.Y. Kao. 1976. Suspended sediment filtration capacity of simulated vegetation. Trans. ASAE, Vol. 19(4):678-682.

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 1976. Storage, treatment, overflow, runoff model, STORM. Computer Program 723-38-L7520. Davis, CA.

U.S. Department of Agriculture. 1963. Soil Conservation Service. national engineering handbook. Washington, D.C.

U.S. Department of Agriculture. 1975. Engineering field manual for conservation practices. Soil Conservation Service, Washington, D.C.

U.S. Department of the Interior, Bureau of Reclamation. 1977. Design of Small Dams. United States Government Printing Office. Washington, D.C.

U.S. Department of the Interior, Office of Surface Mining. 1983. Surface Coal Mining and Reclamation Operations; Permanent Regulatory Program; Siltation Structures. 48 Federal Register, No. 187:44032.

U.S. Environmental Protection Agency. 1975. Process design manual for suspended solids removal. Report EPA 625/1-75-003a. U.S. Environmental Protection Agency. Washington, D.C.

U.S. Environmental Protection Agency. 1976. Erosion and sediment control, surface mining in the eastern U.S. Volume 2: Design. EPA-625/3-76-006.

U.S. Soil Conservation Science. 1972. Hydrology, section 4, soil conservation service national engineering handbook. U.S. Department of Agriculture, Washington, D.C.

U.S. Soil Conservation Service. 1968. Hydraulics of broad crested spillways. Technical release No. 39, U.S. Department of Agriculture; Washington, D.C.

U.S. Soil Conservation Service. 1973. A method for estimating volume and rate of runoff in small watersheds. SCS-TP-149. Washington, D.C.

Wakeman, R.J. 1979. Progress in filtration and separation - 1. Elsevier Scientific Publishing Company, Amsterdam, The Netherlands.

Ward, A.D. 1979. Model studies aid pond design. Water and Wastes Engineering. May, pp. 36-38.

Ward, A.D. and C.T. Haan. 1980. Evaluation of strip mine sediment detention basins in Kentucky. Transactions ASAE. Vol. 23(4):854-858, 865.

Ward, A.D., B.J. Barfield and J.S. Tapp. 1979. Sizing surface mine sediment control reservoirs for theoretical detention time. Proceedings of the Symposium on Surface Mining Hydrology, Sedimentology, and Reclamation. 1979 December 4-7; University of Kentucky, Lexington, Kentucky. pp. 73-82.

Ward, A.D., C.T. Haan and B.J. Barfield. 1977. Simulation of the sedimentology of sediment detention basins. Research report 103. Water Resources Research Institute. University of Kentucky, Lexington, Kentucky.

Ward, A.D., C.T. Haan and J.S. Tapp. 1979. The DEPOSITS sedimentation pond design manual. University of Kentucky, Lexington, Kentucky.

Ward, A.D., D.L. Rausch, C.T. Haan and H.G. Heinemann. 1981. A verification study on a reservoir sediment deposition model. Transactions ASAE Vol. 24(2):340-346, 352.

Ward, A.D., R.G. Williams, and C.T. Haan. 1979. Small scale modelling of Reservoir Sedimentation. Paper presented at ASCE Convention, April 2-6. Bonton, Mass.

Watson, L.E., R.W. Parker and D.F. Polster. 1980. Manual of species suitability for reclamation. Alberta Land Conservation and Reclamation Council Report #RRTAC 80-5. 2 vols. 541 pp.

Whittaker, H. 1983. Water recovery from drilling fluids using UF/RO membranes. Environmental Protection Service. Spill Technology Newsletter. September-October 1983, p. 109-112.

Williams, J.R. 1975. Sediment-yield prediction with universal equation using runoff energy factor. Present and Prospective Technology for Predicting Sediment Yields and Sources, ARS-S 40:244-251.

Williams, J.R. 1981. SPNM, a model for predicting sediment, phosphorous and nitrogen yields from agricultural basins. Water Resources Bulletin 16(5):843-848.

Wilson, B.M., B.J. Barfield, and I.D. Moore. 1982. Design manual for the SEDIMOT II hydrology and sedimentology model. A hydrology and sedimentology watershed model. Part 1: Modelling techniques. Lexington, Kentucky. University of Kentucky, Department of Agricultural Engineering. 302 pp.

Wilson, B.M., B.J. Barfield, and R.C. Warner. 1983. Modelling the performance of sediment detention ponds. Volume I: The CSTRS model based on simple reactor theory. No. IMMRB4/093, Institute for Mining and Minerals Research, University of Kentucky, Lexington, Kentucky.

Wilson, B.N. 1984. Modelling sediment deposition, resuspension, mixing and bed degradation in detention ponds. Unpublished Ph.D. dissertation. University of Kentucky, Lexington, KY.

Wilson, B.N., B.J. Barfield, and D.G. Collivier. 1984. Modelling the performance of sediment detention ponds. Vol. II: The basin model based on advection - diffusion and reactor mixing concepts. Report No. IMMR 84/122, Institute for Mining and Minerals Research, University of Kentucky, Lexington, Kentucky.

Wilson, L.G. 1967. Sediment removal from flood water by grass filtration. Trans. ASAE 10:35-37.

Wischmeier, W.H. 1975. Estimate the soil loss equations cover and management factor for undisturbed areas. Present and prospective technology for predicting sediment yields and sources. ARS-540, U.S. Dept. of Agriculture, ARS, pp. 118-124.

Wischmeier, W.H. and D.D. Smith. 1978. Predicting rainfall-erosion losses - a guide to conservation planning. USDA Agriculture Handbook 537. Washington, D.C.

Ziemkiewicz, P.F. (ed.). 1983. Effects of Coal Mining on the Hydrology of Alberta's Eastern Slopes. Alberta Land Conservation and Reclamation Council Report #RRTAC 83-4. 123 p.